Influence of joints on Rock mass strength through modeling

A thesis submitted in partial fulfillment of the
requirements for the degree of

Master of Technology in Civil Engineering

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Under the Guidance of

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2013

CERTIFICATE

This is to certify that the thesis entitled, **" INFLUENCE OF JOINTS ON ROCK MASS STRENGTH THROUGH MODELING "** submitted by **Mr Bishnu Pada Bose** in partial fulfillment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in "Geotechnical Engineering" at the National Institute of Technology, Rourkela is an authentic work carried out by him under our supervision and guidance.

To the best of our knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

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ACKNOWLEDGEMENT

I express my sincere gratitude and sincere thanks to **Prof. N. Roy** and **Prof. C.R Patra** for their guidance and constant encouragement and support during the course of my Research work. I truly appreciate and values their esteemed guidance and encouragement from the beginning to the end of this works, their knowledge and accompany at the time of crisis remembered lifelong.

I sincerely thank to our Director **Prof. S. K. Sarangi**, and all the authorities of the institute for providing nice academic environment and other facility in the NIT campus, I express my sincere thanks to Professor of Geotechnical group **Prof. S. P. Singh**, and **Prof. S. K. Das** for their useful discussion, valuable suggestions and continuous encouragement and motivation. Also I would like to thanks all Professors of Civil Engineering Department those who are directly and indirectly helped us.

I am also thankful to all the staff members of Geotechnical Engineering Laboratory for their assistance and co- operation during the course of experimental works. I also thank all my batch mates who have directly or indirectly helped me in my project work and shared the moments of joy and sorrow throughout the period of Project work finally yet importantly, I would like to thanks my **Late Parents, Shri Beni Madhab Bose** and **Smt Braja Rani Bose** who taught me the value of hard work by their own example.

At last but not the list, I thank to all those who are directly or indirectly associated in completion of this Research work.

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CONTENTS

ABSTRACT

An assessment of the strength and deformational response of jointed rock masses is an essential requirement in the site selection, design and successful execution of Civil and Mining Engineering projects. An attempt has been made in the present study to develop a interrelation between strength and deformability of jointed block masses with the properties of intact specimens, obtained from simple laboratory tests, taking into account the influence of the properties of the joints in various angle and orientation. Jointed rock masses comprise interlocking angular particles or blocks of hard brittle material separated by discontinuity surfaces which may or may not be coated with weaker materials. The strength of such rock masses depends on the strength of the intact pieces and on their freedom of movement which, in turn, depends on the number, orientation, spacing and shear strength of the discontinuities.

Various joint configurations will be introduced to achieve the most common modes of failure occurring in nature. A coefficient called Joint factor has been used to account for the weakness brought into the intact rock by jointing.

Considering the importance of this study the experimental study has been under taken to determine the strength and deformation behavior of jointed rock mass. Models have been prepared using plaster of Paris and plaster of Paris & sand and different degrees of anisotropy have been induced by making joints in them varying from 0 to 90 degree. The specimens were tested under direct shear, uniaxial compression to determine the various parameters. From this study a guidelines would made for assessing probable modes of failure of a jointed mass which will enable one to estimate the relevant strength and tangent modulus of the jointed rock mass.

LIST OF FIGURE

LIST OF TABLE

NOTATIONS

J_f = Joint factor

Jn = Number of joints per meter length.

n = Joint inclination parameter

r = Roughness parameter.

β = Orientation of joint.

σ_{cj} = Uniaxial compressive strength of jointed rock.

σ_{ci} = Uniaxial compressive strength of intact rock

σ_{cr} = Uniaxial compressive ration.

Etj = Tangent modulus of jointed rock

Eti = Tangent modulus of intact rock

Er = Elastic modulus ratio.

τ = Shear strength

υ = angle of friction

c=Cohesion

φ=friction angle

J = Joint

POP= plaster of Paris

UCS= Uniaxial compressive strength

CHAPTER-1

Introduction

Jointed rock behavior depends upon inherent morphological, geological and environmental factors. Rock mass characteristics and behavior depends on whether the rock is intact, jointed, highly jointed or crushed. For the successful solution of problems faced in jointed rock the two important design parameters, namely shear strength and deformation characteristics should be analyzed correctly. Realistic evaluation of shear strength and deformation characteristics presents formidable theoretical and experimental difficulties due to the complex behavior of jointed rock.

A fair assessment of strength and deformational behavior of jointed rock masses is necessary for the design of slopes, foundations, underground openings and anchoring systems. Both the intact rock and the properties of the joints govern the mass response. If the mass is not highly fractured and the joint system has only few sets (say five or less), then the mass usually behaves anisotropically. Surface or near surface activities in rock mass occur under low confining pressure. In such cases the influence of joints is quite predominant. The uncertainty in predicting the behavior of a jointed mass under uniaxial stress is essentially caused by scale effects and the unpredictable nature of the modes of failure. Extensive field tests are often required to assess the strength and deformability of the ground making the exercise quite expensive. To minimize this uncertainty an extensive experimental study has been carefully planned and executed to develop a more reliable link between the strength and modulus of jointed rock masses and those of the intact rock. In order to understand the behavior of jointed rock masses, it is necessary to start with the components which go together to make up the system the intact rock material and individual discontinuity surfaces. Depending upon the number, orientation and nature of the discontinuities, the intact rock pieces will translate, rotate or crush in response to stresses imposed upon the rock mass. The strength and deformation behavior of rock mass is governed by both intact rock properties and properties of discontinuities.

The strength of rock mass depends on several factors as follows:

- The angle made by the joint with the principal stress direction.
- The degree of joint separation.
- Opening of the joint
- Number of joints in a given direction
- Type and influence of Gouge filling material
- Thickness of the gouge fill material
- Spacing of joint
- Strength along the joint
- Joint frequency
- Joint roughness

Since there are a large number of possible combinations of block shapes and sizes, it is obviously necessary to find any behavioral trends which are common to all of these combinations. The establishment of such common trends is the most important objective of this study.

CHAPTER-2

Literature review

Yaji (1984) conducted triaxial tests on intact and single jointed specimens of plaster of Paris, sandstone, and granite. He has also conducted tests on step-shaped and berm-shaped joints in plaster of Paris. He presented the results in the form of stress strain curves and failure envelopes for different confining pressures. The modulus number K and modulus exponent n is determined from the plots of modulus of elasticity versus confining pressure. The results of these experiments were analyzed for strength and deformation purposes. It was found that the mode of failure is dependent on the confining stress and orientation of the joints. Joint specimens with rough joint surface failed by shearing across the joint, by tensile splitting, or by a combination of thereof.

Arora (1987) carried out several tests on intact and jointed rock specimens of plaster of Paris, Jamarani sandstone, and Agra sandstone. Extensive laboratory testing of intact and jointed specimens in uniaxial and triaxial compression revealed that the important factors which influence the strength and modulus values of the jointed rock are joint frequency, joint orientation with respect to major principal stress direction, and joint strength. Based on the results he defined a joint factor (J_f) as, $J_f = Jn/(n * r)$

Where, Jn = number of joints per meter depth ,

n = Inclination parameter depending on the orientation of the joint ,

r = roughness parameter depending on the joint condition

Singh et al. (2002) executed several experiments in the laboratory. The experiments were conducted on specimens of a jointed block mass formed of saw cut blocks of a model material (sand, lime brick). The joint configuration was varied to achieve the possible modes of failure commonly occurring in the field. The findings of study established four distinct modes of failure namely splitting of intact material, shearing of intact material, rotation of blocks and sliding along the critical joints. These modes of failure have been found to be dependent on the configuration of joints and interlocking conditions. Guidelines have been suggested to assess the probable modes of failure in the field based on the mapping of joints.

A weakness coefficient called Joint Factor has been used to describe the effect of Jointing introduced in the intact rock. Expressions have been suggested to compute the strength and tangent modulus of the jointed mass through the Joint Factor. The methods to compute the Joint Factor for various modes of failure have also been established.

Sitharam and Latha (2002) in this paper, a practical equivalent continuum model presented by Sitharam et al. is used for the analysis of excavations in rock masses. In this model, the rock mass properties are represented by a set of empirical relations, which express the elastic modulus of jointed rock mass as a function of joint factor and the elastic modulus of intact rock. Elastic modulus of a rock is determined from its stress-strain curve as the tangent modulus at 50% axial strain. Extensive laboratory testing of intact and jointed specimens of different grades of plaster of Paris, sandstone and granite revealed that the properties of rock joint can be integrated into a single entity called Joint factor given by the following equation, $J_f = Jn/(n*r)$

Where, Jn = number of joints per meter depth

n = inclination parameter depending on the orientation of the joint

r = roughness parameter depending on the joint condition.

Yilmat and Sendir (2002) this study aims to express the relationships between Schmidt rebound number (N) with unconfined compressive strength (UCS) and Young's modulus (Et) of the gypsum by empirical equations. As known, the Schmidt hammer has been used worldwide as an index test for a quick rock strength and deformability characterisation due to its rapidity and easiness in execution, simplicity, and portability, low cost and non destructiveness. The tests include the determination of Schmidt hammer rebound number (N), tangent Young's modulus (Et) and unconfined compressive strength(UCS) and made a relationship between UCS–Et –N were performed and derived as expressed by empirical equations of UCS = exp(0.818 + 0.059 N) and Et = exp(1.146 + 0.054 N).

Jade and Sitharam (2003) studied statistical analysis of the uniaxial compressive strength and of the elastic modulus of jointed rock masses under different confining pressures. Properties of the rock masses with different joint fabric, with and without gouge have been considered in the analysis. A large amount of experimental data of jointed rock masses from the literature has been compiled and used for this statistical analysis. The uniaxial compressive strength of a rock mass has been represented in a non-dimensional form as the ratio of the compressive strength of the jointed rock to the intact rock. In the case of the elastic modulus, the ratio of elastic modulus of jointed rock to that of intact rock at different confining pressures is used in the analysis. The effect of the joints in the rock mass is taken into account by a joint factor. The joint factor is defined as a function of joint frequency, joint orientation, and joint strength. Several empirical relationships between the strength and deformation properties of jointed rock and the joint factor have been arrived at via statistical analysis of the experimental data. A comparative study of these relationships is presented. The effect of confining pressure on the elastic modulus of the jointed rock mass is also considered in the analysis. . The study conclude that the jointed rock mass will act both as an elastic material and a discontinuous mass. The results obtained by the model with equivalent properties of the jointed rock mass predict fairly well the behavior of jointed rock mass.

Singh and Rao (2005) A large number of uniaxial compressive strength (UCS) tests were conducted on the specimens of jointed block mass having various combinations of orientations and different levels of interlocking of joints. Four dominating modes of failure were observed. The findings of the study have been verified by applying it to estimate the ultimate rock mass strength of nine rock types from few dam sites in the lower Himalayas. The ultimate strength obtained by the present methodology is compared with that obtained through the Q classification system. It is concluded that reasonably good estimates on field strength of jointed rocks are possible by using the correlations suggested in this study.

Tiwari and Rao (2006) carried out number of experiment of uniaxial , triaxial and true triaxial on a jointed specimen of model materials made by sand and lime (after Tiwari and Rao, 2004), test criteria was various angle of orientation joint . From the experiment they found that the deformation modulus of rock mass is influenced due to intermediate principal stress similar to enhancement in triaxial compressive strength. The modulus enhancement in rock mass with joint geometries corresponding to Φ=40 and 60 degree is more than in case of joint geometries of Φ=0, 20, 80 and 90 degree. Thus weak rocks are subjected to more modulus enhancement than comparatively harder rocks.

Ebadi et al. (2012) results obtained from the developed analytical model, as shear and normal stresses due to the lateral stresses respectively increase and decrease the modulus of jointed rock mass. The modulus of jointed rock mass increases with the increase of the lateral stresses. Increase of the minimum confining stress (σ_3) and intermediate lateral stress (σ_2) causes an increase of the jointed rock mass modulus.

CHAPTER -3

Some basic concepts

3.1 Engineering description of rock

Geological recognize only one naturally occurring earth materials called rock. Engineering differentiate between rocks and soils, although sometimes the dividing line is unclear. In particular, an engineer differentiates between the reactions of rock and soils to the force imposed on them or in them by construction. The study of the reaction of soils to these forces is called soils mechanics and the study of the reaction of rocks is called rock mechanics.

Both rock and soils are made up of mineral and organic particles. In the former, the particle are generally bonded or cemented together and an initial yield resistance must be overcome before they shear in an unconfined state, and a very small energy input is required to precipitate breakdown

Rock mechanics must therefore be defined as the study of rock deformation and fracture in both its intact material form and as a discontinuous mass. Nevertheless , though convention or otherwise, rocks are usually described for engineering purpose through their action as materials , and it is useful to start by considering some of the simple test to which rocks are subjected and which can be used to define and compare their engineering reactions.

3.2 Discontinuities of rock masses

Discontinuities in a rock mass such as joint, foliation, or faults form a blocks. While the blocks usually consist of competent rock, discontinuities represent zones of weakness .The properties and the behaviors of the discontinuities are therefore critical for the analysis and assessment of block stability.

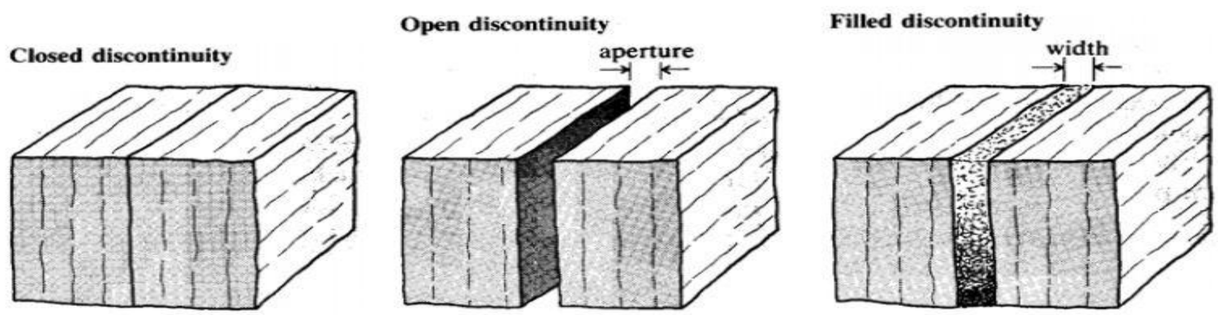


Fig.3.1: Various type of discontinuities of jointed Rock masses

Discontinuities are usually categorized according to the manner in which they were formed. The following are standard definitions of the most commonly encountered types of discontinuities:

a) Fault

A discontinuity along which there has been an observable amount of displacement. Faults are rarely single planar units; normally they occur as parallel or sub-parallel sets of discontinuities along which movement has taken place to a greater or less extent.

b) Bedding plane

This is surface parallel to the surface of deposition, which may or may not have physical expression. Note that the original attitude of the bedding plane should not be assumed to be horizontal.

c) Foliation

Foliation is parallel orientation of platy minerals, or minerals banding in metamorphic rock.

d) Joint

A joint is a discontinuity in which there has been no observable relative movement. A series of parallel joint is called a joint set; two or more intersecting sets produced a joint system. Two sets of joint approximately at right angle to one another are said to be orthogonal. Joints are the most common discontinuity in rock and generally contribute significant effect on the rock mass behavior. Joints are breaks of geological origin along which there has been no visible displacement (Park, 1989). Joint may be formed in a systematic way (fracture occur in sub parallel joint or irregular geometry) or non-systematic way (non-parallel joint or irregular geometry). Joints are found in all competent rocks within about 1 km of the earth's surface, at all orientations and at sizes ranging from a few millimeters to several hundred meter. They may be intact, open, filled or healed.

❖ Properties of Discontinuities

This section discuss briefly on the most important aspect of those properties of discontinuities that influenced the engineering behavior of rock mass. Spacing is the perpendicular distance between adjacent discontinuities; ad is usually expressed as the mean spacing of a particular set of joints. Spacing determines the sizes of the block making up the rock mass. The mechanism of the deformation and the failure can vary with the ratio of discontinuity spacing to excavation size. If the joint spacing is very much smaller than the width of excavation instability will prevail. Aperture is the perpendicular distance separating the adjacent rocks walls of an open discontinuity in which the intervening space is filled with air or water. Aperture is thereby distinguished from the width of a filled discontinuity. Jointed rock masses at depth, apertures will be small, probably less than half a millimeter. The apertures of real discontinuities are likely to vary widely over the extent of the discontinuity. Clearly, variation of aperture will have an influenced on the shear strength of the discontinuity. More important is the influence of aperture on the permeability or hydraulic conductivity of the discontinuity of the rock mass.

3.3 Jointed rock masses

Jointed rock masses comprise interlocking angular particles or blocks of hard brittle material separated by discontinuity surfaces which may or may not be coated with weaker materials. The strength of such rock masses depends on the strength of the intact pieces and on their freedom of movement which, depends on the number, orientation, spacing and shear strength of the discontinuities.

Fig:3.2: Jointed rock mass

3.4 Concept of Joint Factor

The influence of jointing on the response of intact rock can be studied through a weakness coefficient called Joint Factor (Ramamurthy, 1993; Ramamurthy and Arora, 1994). This coefficient reveals the ‘‘weakness’’ brought in to the intact rock through jointing and takes into account the combined effect of frequency of joints, their inclination and roughness along the critical joints. The higher the Joint Factor, the greater is the ‘‘weakness’’. It is defined taking the three key factors controlling the response of the jointed mass into account.

$J_f = J_n / n_* r$

Where J_f =Joint factor

Jn = number of joints/m depth in the direction of loading

n = critical joint inclination parameter presented in table. The parameter was derived by conducting experiments on specimens with inclined joints (Ramamurthy, 1993; Ramamurthy and Arora, 1994).

r = sliding joint strength parameter =tanΦj

Where Φj is friction angle along the critical joint at sufficiently low normal stress so that the initial roughness of the surface is reflected through this value.

Table.3.1: Values of inclination parameter (n) with respect to orientation angle (β)

Orientation of joint (β)	Inclination parameter (n)	Orientation of joint (β)	Inclination parameter (n)
0	0.810	50	0.306
10	0.460	60	0.465
20	0.105	70	0.634
30	0.046	80	0.814
40	0.071	90	1.000

3.5 Deformation behavior of jointed rock masses

Deformation behavior of jointed rock is greatly influenced by deformability along the joints. In addition to significant influence on strength of the rocks joints will generally lead to marked reduction in the deformation modulus which is another parameter of interest to the designer. In situ testing such as plate load and radial jacking have been generally performed in practice for determining the rock mass module values. The deformation characteristic of a rock mass depends on the orientation of joint with respect to the loading direction, the insitu stress condition, the spacing of joints and the size of loading region

Equation given by Konder (1963),

$$(\varepsilon_1)/(\sigma_1 - \sigma_3) = a + b\varepsilon 1$$

Where ε_1 = axial strain, a= reciprocal strain modulus, b= reciprocal of asymptotic value of deviator stress.

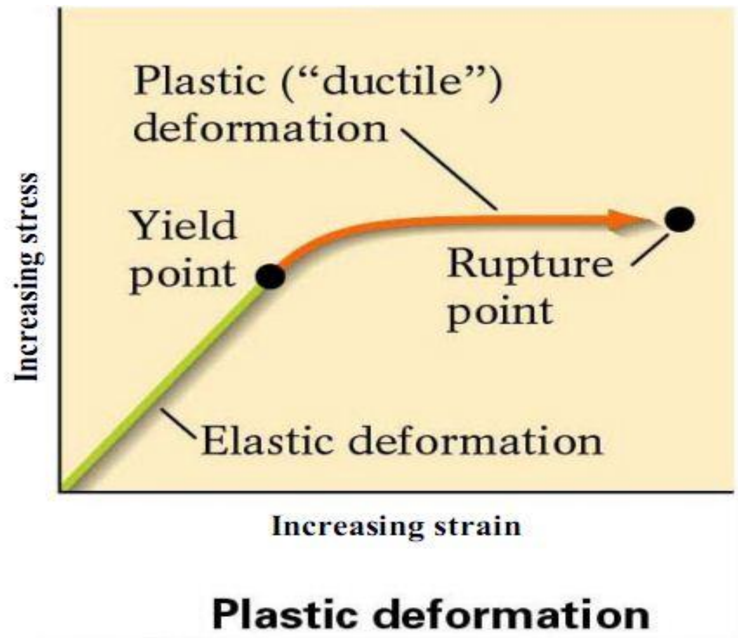


Fig. 3.3 : Plastic deformation curve of rock mass

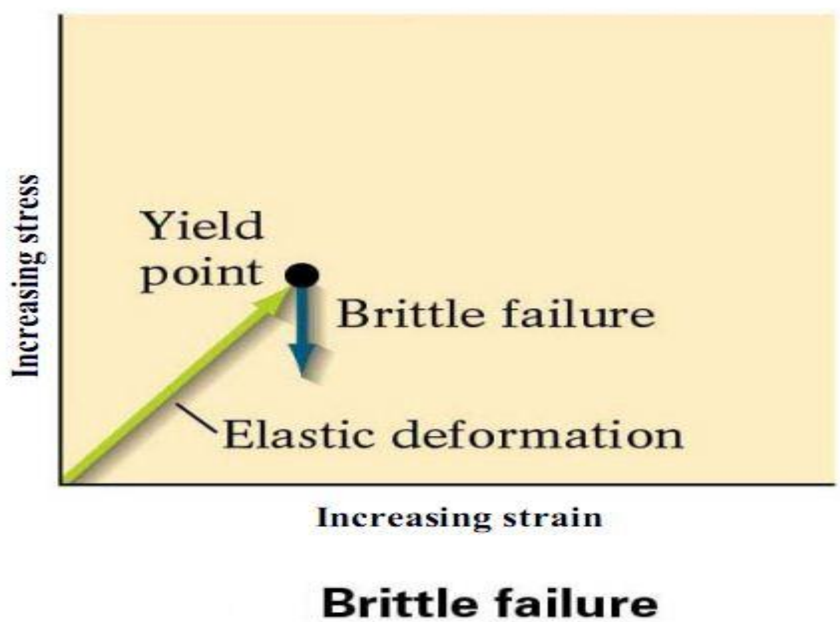


Fig. 3.4: Brittle deformation curve of rock mass

3.6 Gypsum Plaster

Plaster of Paris is a type of building material based on calcium sulphate hemihydrates, nominally $CaSO_4.0.5H_2O$. It is created by heating Gypsum to about 300ºF (150ºC).

$$CaSO_4.H_2O \longrightarrow 2CaSO_4.0.5H_2O + 3H_2O$$

A large Gypsum deposit at Montmartre in Paris is the source of the name. When the dry plaster powder is mixed with water, it reforms into Gypsum. plaster is used as a building material similar to mortar or cement. Like those material Plaster starts as a dry powder that is mixed with water to form a paste which liberates heat and then hardens. Unlike mortar and cement, Plaster remains quite soft after setting and can be easily manipulated with metal tools or even sand paper. These characteristics make Plaster suitable for a finishing, rather than a load bearing material.

3.7 X-Ray Diffraction Analysis

X-Ray powder Diffraction analysis is a powerful method by which X-Rays of a known wavelength are passed through a sample to be identified in order to identify the crystal structure. The wave nature of the X-Rays means that they are diffracted by the lattice of the crystal to give a unique pattern of peaks of 'reflections' at differing angles

and of different intensity, just as light can be diffracted by a grating of suitably spaced lines. The X-ray diffraction (XRD) test was used to determine the phase compositions of plaster of Paris. The basic principles underlying the identification of minerals by XRD technique is that each crystalline substance has its own characteristics atomic structure which diffracts x-ray with a particular pattern. In general the diffraction peaks are recorded on output chart in terms of 2θ, where θ is the glancing angle of x-ray beam. The 2θ values are then converted to lattice spacing „d" in angstrom unit using Bragg"s law, $\mathbf{d = \lambda/2n \sin \theta}$; where n is an integer & λ = wave length of x-ray specific to target used. The X-Ray detector moves around the sample and measures the intensity of these peaks and the position of these peaks [diffraction angle 2θ]. The highest peak is defined as the 100% * peak and the intensity of all the other peaks are measured as a percentage of the 100% peak.(Fig. 5.1)

3.8 SEM/EDX Analyses

A SEM (Scanning Electron Microscope) can be utilized for high magnification imaging of almost all materials. With SEM in combination with EDX (Energy Dispersive X-ray Spectroscopy), it is also possible to find out which elements are present in different parts of a sample. The microstructures of plaster of Paris were studied. Micro-photographs of the sample are shown in fig. (5.2-5.4). It is clearly observed that most of the particles are almost angular structure with irregular surfaces.

3.8 Elastic Modulus

Elastic modulus expressed as the tangent modulus at 50% of stress failure is considered in this analysis. The elastic modulus ratio is expressed as

$E_r = E_{tj} / E_{ti}$

Where,

E_r = Elastic modulus ratio

E_{tj} =is the tangent modulus of jointed rock

E_{ti} =is the tangent modulus of intact rock.

3.9 Uniaxial compressive strength ratio

Just as in concrete design the major criterion for specification is cube strength, so in rock mechanics the most quoted index of mechanical behavior is unconfined compressive strength.

The uniaxial compressive strength of intact and jointed rock mass is represented as the ratio of load / area unit is N/Sqmm or MPa. The uniaxial compressive strength ratio is expressed as:

$$\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$$

Where, σ_{cj} = uniaxial compressive strength of jointed rock; σ_{ci} = uniaxial strength of intact rock. The uniaxial compressive strength of the experimental data should be plotted against the joint factor .The joint factor for the experimental specimen should be estimated based on the joint orientation, strength and spacing. Based on the statistical analysis of the data, empirical relationship for uniaxial compressive strength ratio as function joint factor (J_f) are derived.

3.10 Applications of rock mechanics

- Tunnel for hydropower intake water
- Tunnel for Transportation systems
- Dam foundation and abutment
- Long wall mining
- To support massive & heavy civil structures

Fig.3.5: Tunnel for hydropower intake water

Fig.3.6: Long wall mining by rock masses

3.11 Strength criterion for anisotropic rocks

- Strength criterion

Unlike isotropic rocks, the strength criterion for anisotropic rocks is more complicated because of the variation in the orientation angle β. A number of empirical formulae have been proposed like by Navier –coulomb and Griffith criteria. It is clearly shown that the strength for all rocks is maximum at β=0º or 90º and is minimum at β=20º or 30º.

- Influence of single plane of weakness

In a laboratory test the orientation of the plane of weakness with respect to principal stress directions remains unaltered. Variation of the orientation of this plane can only be achieved by obtaining cores in different directions. In field situation, either in foundation of dams around underground or open excavation, the orientation of joint system remains stationary but the directions of principal stress rotate resulting in a change in the strength of rock mass. Jaegar and Cook (1979) developed a theory to predict the strength of rock Containing a single plane of weakness,

$$\sigma_1 - \sigma_3 = (2c + 2\,\sigma_3 \tan\phi)/(1 - \tan\phi.\cot\beta)\sin2\beta$$

Where, ϕ = friction angle; β = Angle of inclination of plane of weakness with vertical failure

σ_1 and σ_3 = major and minor principal stresses, while sliding will occur for angles 0º to 90º .

3.12 Parameters characterizing type of anisotropy

Broadly three possible parameters define the concept of strength anisotropy of rocks. These are

1) Location of maximum and minimum compressive strength (σ_{cj}) in the anisotropic curve in terms of the orientation angle (β).

2) The value of uniaxial compressive strength at these orientations

3) General shape of anisotropy curve.

Rock exhibit maximum strength at 0˚ or 90˚and minimum strength between 20˚ to 40˚(Arora and Ramamurthy 1987) has introduced an inclination parameter (n) to predict the behavior of different orientation of joints in rock behavior. The relationship between n and β is given on the experiment on test specimen. The variation n and β was observed to be similar to the variation of uniaxial compressive strength ratio σ_{cr} with the value for the corresponding β values.

3.13 Failure modes in rock mass

The failure modes were identified based on the visual observations at the time of failure. The failure modes obtained are:

(i) Splitting of intact material of the elemental blocks,

(ii) Shearing of intact block material,

(iv) Rotation of the blocks, and

(v) Sliding along the critical joints.

These modes were observed to depend on the combination of orientation n and the stepping. The angle θ in this study represents the angle between the normal to the joint plane and the loading direction, whereas the stepping represents the level/extent of interlocking of the mass. The following observations were made on the effect of the orientation of the joints and their interlocking on the failure modes. These observations may be used as rough guidelines to assess the probable modes of failure under a uniaxial loading condition in the field.

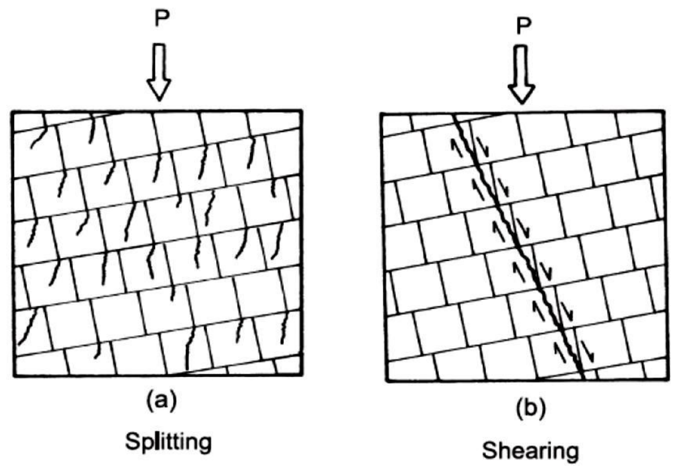


Fig.3.7: Splitting and shearing modes of failures in rocks

❖ Splitting

Material fails due to tensile stresses developed inside the elemental blocks. The cracks are roughly vertical with no sign of shearing. The specimen fails in this mode when joints are either horizontal or vertical and are tightly interlocked due to stepping.

❖ Shearing

In this category, the specimen fails due to shearing of the elemental block material. Failure planes are inclined and are marked with signs of displacements and formation of fractured material along the sheared zones. This failure mode occurs when the continuous joints are close to horizontal (i.e., $\theta \leq 10^\circ$) and the mass is moderately interlocked.

As the angle n increases, the tendency to fail in shearing reduces, and sliding takes place. For

$\theta \approx 30^\circ$, shearing occurs only if the mass is highly interlocked due to stepping.

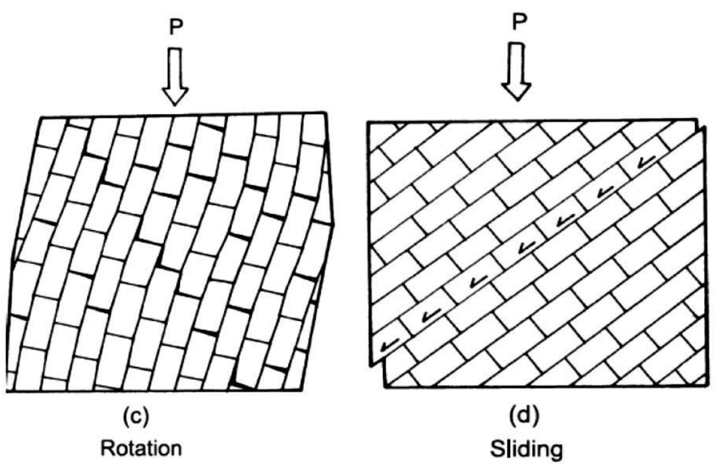


Fig.3.8 : Sliding and rotation modes of failure

- Sliding

The specimen fails due to sliding on the continuous joints. The mode is associated with large deformations, stick–slip phenomenon, and poorly defined peak in stress– strain curves. This mode occurs in the specimen with joints inclined between $\theta \approx 20^{\circ}$– 30° if the interlocking is nil or low. For orientations, $\theta = 35^{\circ}$– 65° sliding occurs invariably for all the interlocking conditions.

- Rotation

The mass fails due to rotation of the elemental blocks. It occurs for all interlocking conditions if the continuous joints have $\theta > 70^{\circ}$, except for θ equal to 90° when splitting is the most probable failure mode.

Table.3.2: Strength of jointed and intact rock mass (Ramamurthy And Arora, 1993)

Class	Description	UCS, MPa
A	Very high strength	>250
B	High strength	100–250
C	Moderate strength	50–100
D	Medium strength	25–50
E	Low strength	5–25
F	Very low strength	<5

Table.3.3: Modular ration classification of intact and jointed rocks(Ramamurthy and Arora 1993)

Class	Description	Modulus ratio, M_{rj}
A	Very high modulus ratio	>500
B	High modulus ratio	200–500
C	Medium modulus ratio	100–200
D	Low modulus ratio	50–100
E	Very low modulus ratio	<50

CHAPTER -4

Laboratory investigation

4.1 Materials used

Experiments have been conducted on preparation two type soft rock specimen by a) plaster of Paris and b) plaster-sand mix, so as to get uniform, identical or homogenous specimen in order to understand the failure mechanism, strength and deformation behavior. It is observed that plaster of Paris has been used as model material to simulate weak rock mass in the field. Many researchers have used plaster of Paris because of its ease in casting, flexibility, instant hardening, low cost and easy availability. Various joint can be made by plaster of Paris. And in the field sand is the one of the composition of many soft rock materials .To obtained strength and deformed abilities in relation to actual rocks has made by POP and sand is one of the suitable material for preparation a soft rock model in geotechnical engineering and hence it is used to prepare models for this investigation.

4.2 Model of the specimens

Two types of model materials (specimen) prepared.

1) Plaster of Paris

2) Plaster of Paris and sand

Type of mould is cylindrical (L/D ratio=2)

On the basis of trial proportion of POP and fine sand by weight will be considered are as follows.

POP: sand = 8:2

Size of each specimen (L/D = 2:1) D = 38 mm and L = 76 mm.

Water quantity has been considered as per the OMC determinations.

4.3 Preparations of specimens

Plaster of Paris is procured from the local market and sand were collected from River Koyel near to NIT Campus . plaster of Paris powder is produced by pulverizing partially burnt gypsum which is duly white in colour with smooth feel of cement. The water content at which maximum density is to be achieved is found out by conducting number of trial tests with different percentage of distilled water. The optimum moisture content was found out to be 30% and 29% by weight for POP & POP-sand mix specimens respectively .For preparation of POP specimen, 132 gm of plaster of Paris is mixed thoroughly with 39.6 cc (30% by weight) water and for POP-sand mix specimen, 135 gm of materials (sand-29 gm. +POP-106 gm.) is mixed thoroughly with 39.15 cc (29% by weight) water to form a uniform paste. The specimens are prepared by pouring the plaster mix in the mould and vibrating on the vibrating table machine for approximately 2 min for proper compaction and to avoid presence of air gaps. After that it is allowed to set for 5 min. and after hardening, the specimen was extruded manually from the mould by using an extruder. The specimens are polished by using sand paper. The polished specimens are then kept at room temperature for 48 hours.

The following standards have been suggested by I.S.R.M committee on Laboratory Test (1972) for compressive strength test:

- The ends of the specimen shall be flat to 0.02 mm (0.0008 inch)
- The ends of the specimen shall be perpendicular to the axis of the specimen within .001 radian(3.5 minute)
- The sides of the specimen shall be smooth and free of abrupt irregularities and straight to within 0.3 mm (0.012 in) over the full length of the specimen.
- The number of specimens to be tested depends on the variability of the results and the desired accuracy and reliability of the mean value. Ten or more specimen are preferable to determine the strength of rocks

4.4 Curing

After keeping the specimens in room temperature 48 hour, they are placed inside desiccators containing a solution of concentrated sulphuric acid (47.7cc) mixed with distilled water (52.3cc). This is done mainly to maintain the relative humidity in range of 40% to 60%. Specimens are allowed to cure inside the desiccators till constant weight is obtained (about 20 days). Before testing each specimen of Plaster of Paris obtaining constant weight dimensioned to L/D = 2:1,at L = 76 mm, D = 38 mm.

4.5 Making joints in specimens

The following instruments are used in making joints in specimen

1) “V” block

2) Light weight hammer

3) Chisel

4) Scale

5) Pencil

6) Protractor

Two longitudinal lines are drawn on the specimen just opposite to each other. At the centre of the line the desired orientation angle is marked with the help paper template by pencil. Then this marked specimen is placed on the “V” block and with the help of chisel keeping its edge along the formed comes under a category of rough joint. The uniaxial compressive strength test and direct shear test are conducted on intact specimens, jointed specimens with single and double joints to know the strength as well as deformation behavior of intact and jointed rocks and the shear parameters respectively.

4.6 experimental setup and test procedure

In this study, specimens were tested to obtain their uniaxial compressive strength, deformation behavior and shear parameters. The tests conducted to obtain these parameter were direct shear test, uniaxial compression test . These tests were carried as per ISRM and IS codes. A large number of uniaxial compressive strength tests were conducted on the prepared specimens of jointed block mass having various combinations of orientations and different levels of interlocking of joints for obtaining the ultimate strength of jointed rock mass.

4.7 Direct Shear test

The direct shear test was conducted to determine (roughness factor) joint strength, r = tanϕj in order to predict the joint factor J_f (Arora 1987). These test were carried out on conventional direct shear test apparatus (IS: 1129- 1985) with certain modifications required for placement of specimens inside the box. Two identical wooden blocks of sizes 59X59X12 mm each having circular hole diameter of 39 mm at the Centre were inserted into two halves of shear box the specimen is then place inside the shear box (60 x 60 mm).

The cylindrical specimen broken into the two equal parts was fitted into the circular hole of the wooden blocks, so that the broken surface match together and laid on the place of shear i.e. the Contact surface of two halves of the shear box.

- Direct shear test equipment

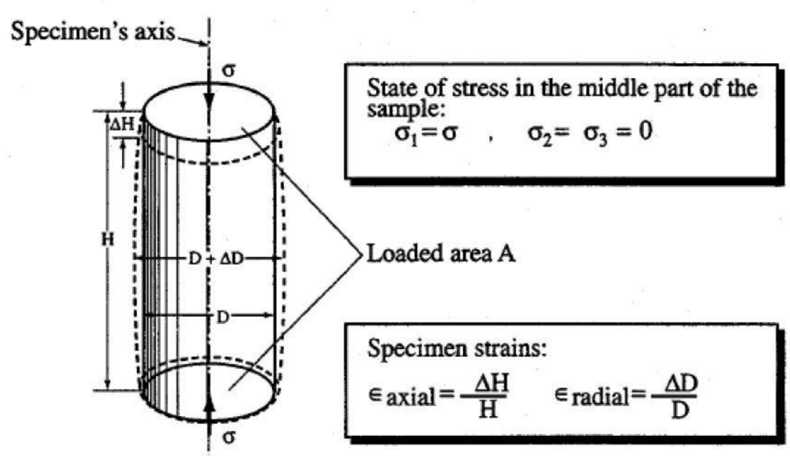
  Proving ring No-099

  Capacity = 2.5 kN

  1 Div or LC = 3.83 N

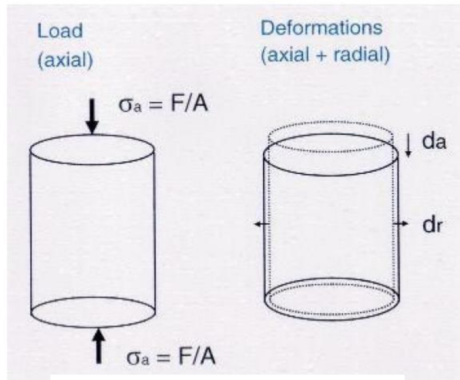
4.8 Uniaxial compressive test:

In Uniaxial Compressive Strength test the cylindrical specimens were subjected to major principal stress till the specimen fails due to shearing along a critical plane of failure. In this test the samples were fixed to cylindrical in shape, length 2 times the diameter, ends maintained flat within 0.02mm. Perpendicularity of the axis were not deviated by 0.001radian and the specimens were tested within 30days. The prepared specimens(L=76 mm, D=38 mm) were put in between the two steel plates of the testing machine and load applied at the predetermined rate along the axis of the sample till the sample fails.. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase of strain. The applied load at the point of failure was noted. The load is divided by the bearing surface of the specimen which gives the Uniaxial compressive strength of the specimen.



STRESSES IN A UCS SPECIMEN

Fig . 4.1 Stresses in UCS specimen



LOADING ON THE SPECIMEN.

Fig . 4.2 Loading on the specimen

- UCS test equipment

  Proving ring no- 1004

  Capacity = 20 kN

  1 Div or LC = 24.242 N

  Dial gauge least count = .01mm

4.9 Parameters studied

The main objective of the experimental investigation has to study the following aspects.

1. UCS test of the intact specimen
2. UCS test of the jointed specimen
3. Direct shear test to know cj and Φj values.
4. Effect of joint factor in the strength characteristic of jointed specimen.
5. Deformation behavior of jointed specimen.
6. Relation between modulus ratio , strength ratio and joint factor.
7. To link joint factor with the strength ratio of jointed to intact rock mass
8. To link joint factor with the modulus ratio of jointed to intact rock mass
9. Strength classification.

Uniaxial compressive strength tests were conducted on intact specimens, jointed specimens with single and double joints to know the strength as well as the deformation behavior of intact and jointed specimen. The jointed specimens were tested for different orientation angles such as 0,10,20,30,40,50,60,70,80,90 degrees. The jointed specimens were placed inside a rubber membrane before testing of UCS to avoid slippage along the joints just after application of the load.

4.10. Types of joints studied

Table. 4.1: Types of joint studied for uniaxial compressive strength both single and double joints

Types of joints with major principal axis for single joint specimens	1J-0º	1J-10º	1J-20º	1J-30º	1J-40º	1J-50º	1J-60º	1J-70º	1J-80º	1J-90º
Types of joints with major principal axis for double joint specimens		2J-10º	2J-20º	2J-30º	2J-40º	2J-50º	2J-60º	2J-70º	2J-80º	2J-90º

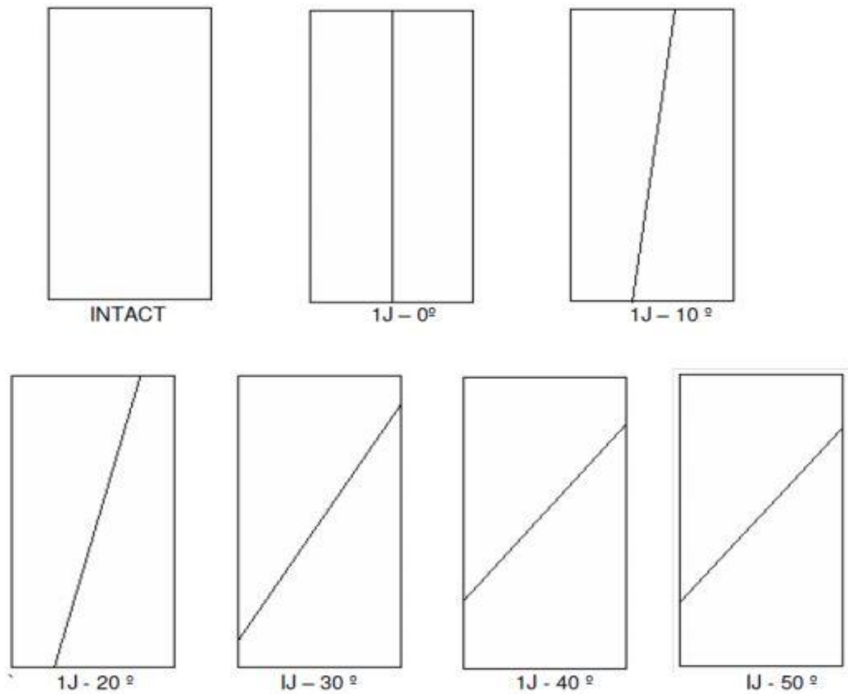


Fig: 4.3: Types of joints studied in plaster of Paris specimens.(some single jointed specimens are shown here)

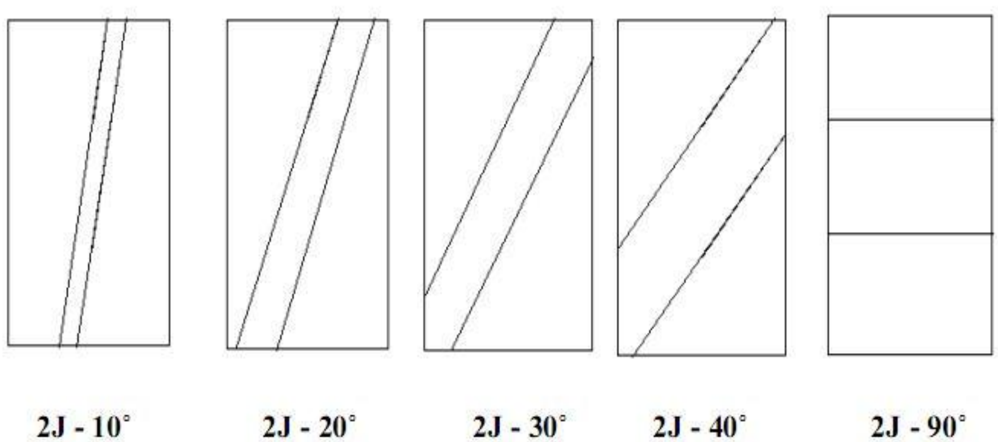
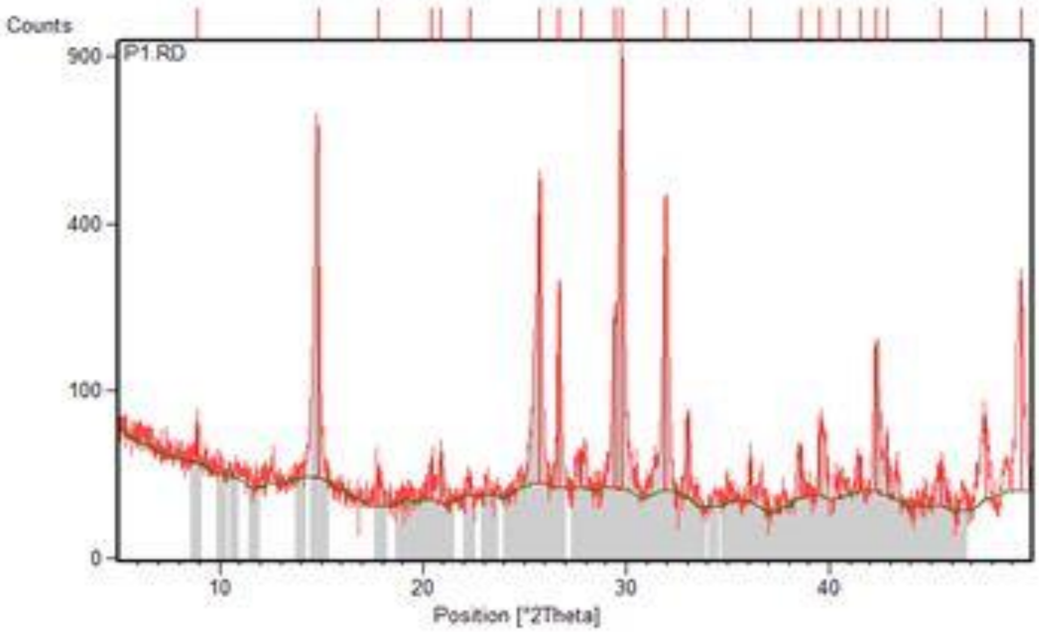


Fig:4.4: Types of joints studied in plaster of Paris specimens.(some double jointed specimens are shown here)

CHAPTER-5

Result and discussions

5.1 Results from XRD, SEM and EDX:



Accepted Ref. Pattern: 83-0438 Scale factor: 0.819

No.	Visib.	Ref. Code	Score	Compound Name	Chemical Formula	Scale Factor	D
1	✔	83-0438	69	Calcium Sulfate Hydrate	Ca (S O4) (H2 O)0.5	0.819	
2	✔	32-0415	43	Graphite Hydrogen Nitrate	C16 H N O3	0.138	
3	✔	29-0127	24	Antimony Fluoride Iodide	Sb I F12	0.230	
4	✔	82-1562	27	Silicon Oxide	Si O2	0.381	
5	✔	31-0261	23	Scawtite	Ca7 (Si6 O18) (C O3)	0.311	
6	✔	24-0027	28	Calcite	Ca C O3	0.188	
7	✔	17-0912	27	Calcium Oxide	Ca O	0.381	
8	✔	33-1161	48	silica	Si O2	0.181	
9	✔	79-1906	42	Quartz	Si O2	0.155	
10	✔	05-0428	39	Sulfur Oxide	S O2	0.475	

Position [°2 Theta]

Fig. 5.1 Microscopic pattern of plaster of Paris

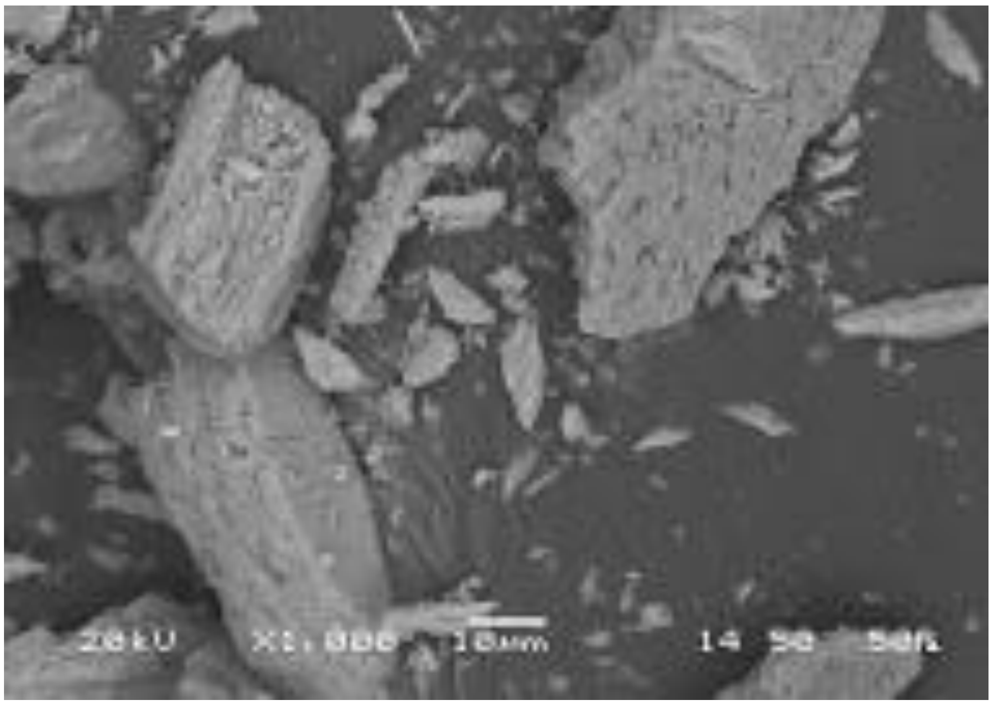


Fig. 5.2: Microstructure of plaster of Paris (X1000)

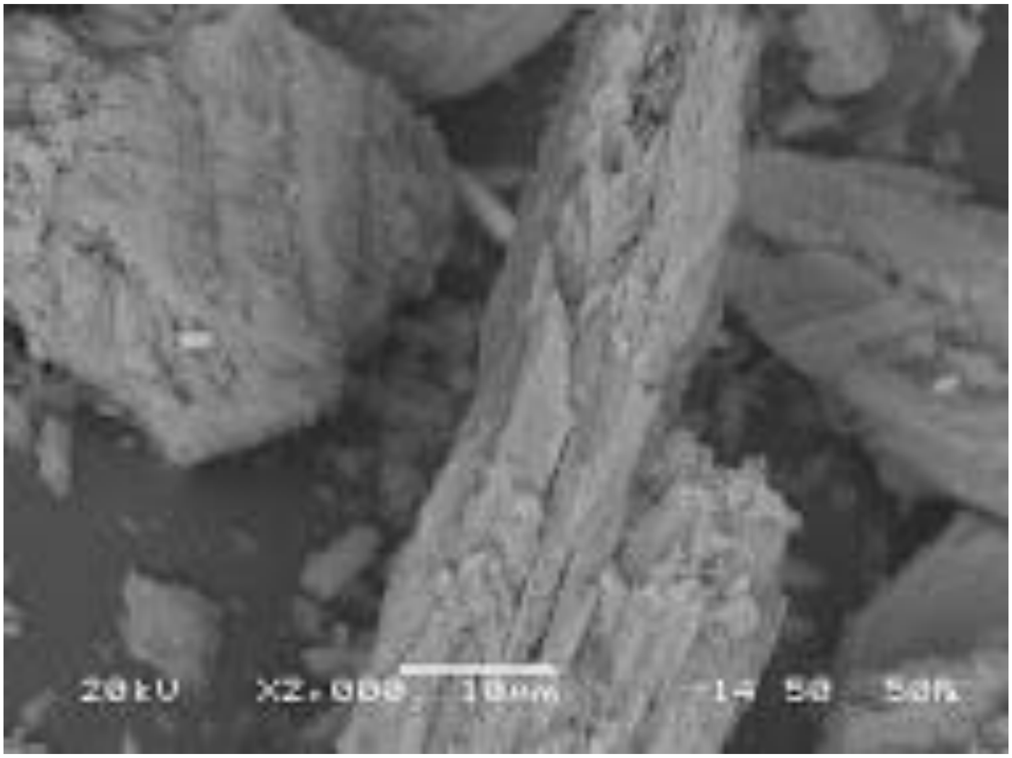


Fig. 5.3: Microstructure of plaster of Paris (X2000)

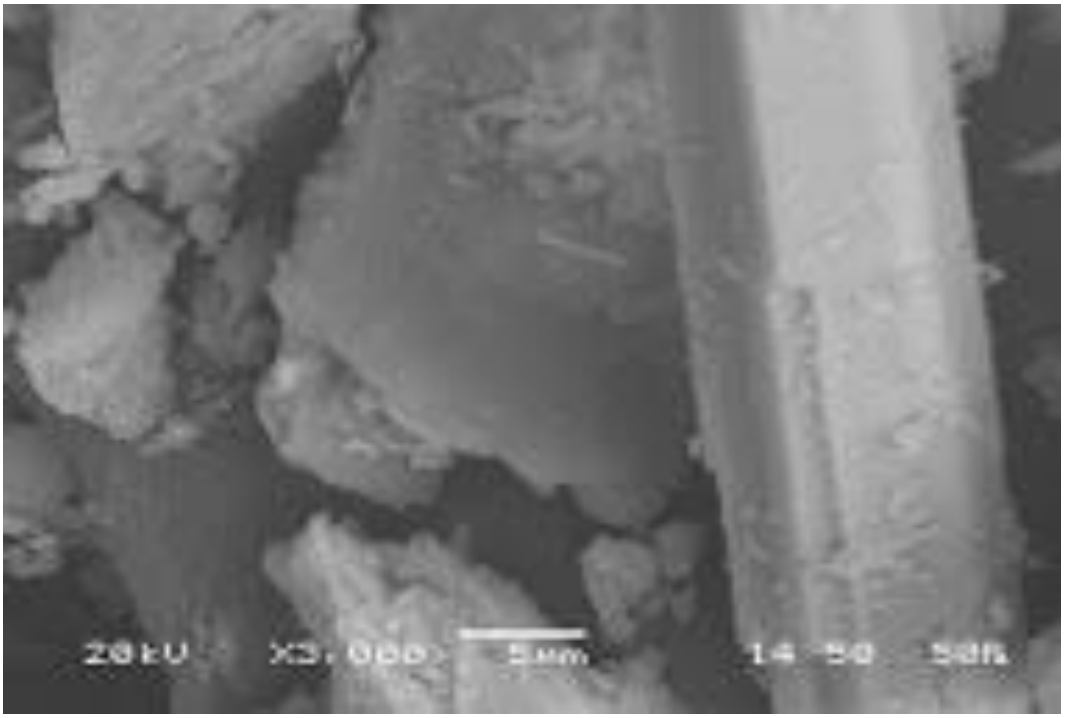


Fig. 5.4: Microstructure of plaster of Paris (X3000)

5.2 Direct shear test results of POP test specimen

The roughness parameter (r) which is the tangent value of the friction angle (Φj) was obtained from the direct shear test conducted at different normal stresses. The value of cohesion (Cj) for jointed specimens of plaster of Paris has been found as 0.178 MPa and value of friction angle (Φj) found as 39º. Hence the roughness parameter (r = tanΦj) comes to be 0.809 for the specimens of plaster of Paris tested.

Table.5.1: Values of shear stress for different values of normal stress on jointed specimens of plaster of Paris in direct shear stress test.

Cross sectional area of samples=1134mm^2

Normal stress ,σ_n (MPa)	Shear stress, τ (MPa)
0.049	0.298
0.098	0.417
0.147	0.537

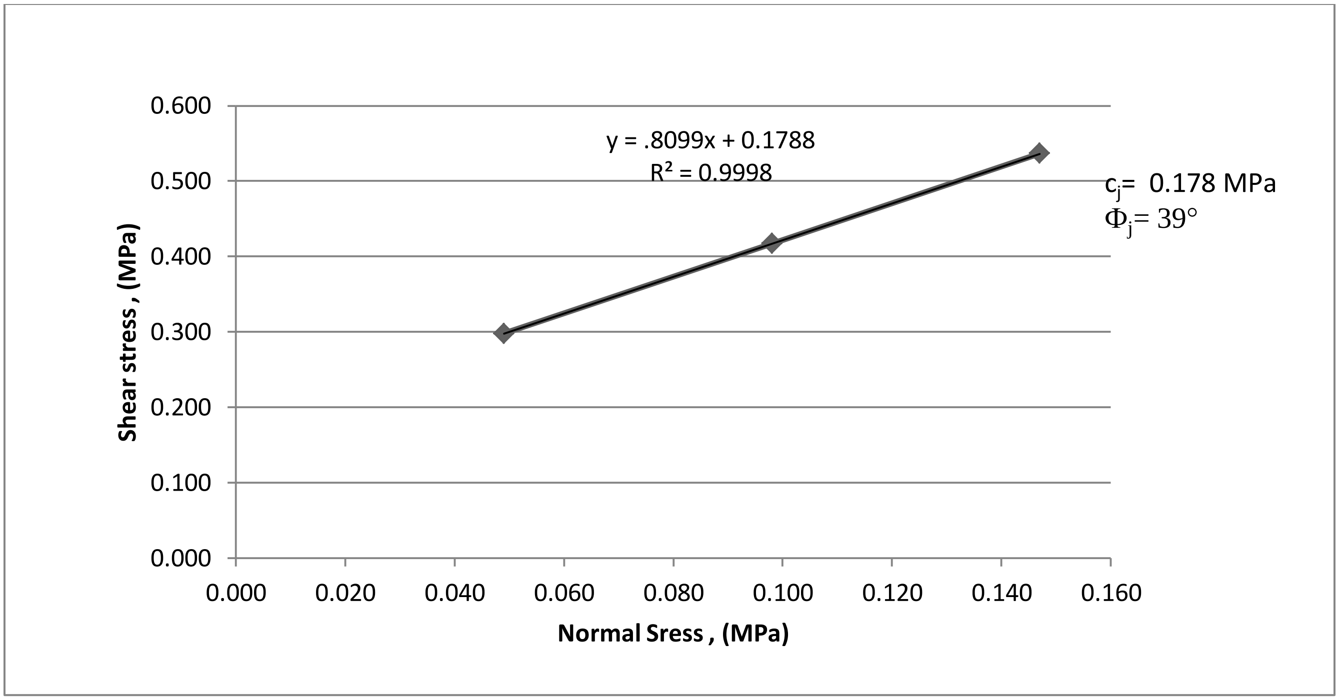


Fig. 5.5: Normal stress vs. shear stress of POP jointed specimen

5.3 Uniaxial compression test results of POP intact specimen:

The variations of the stress with strain as obtained by uniaxial compression strength test for the intact specimen of plaster of Paris is and its corresponding stress Vs. strain values are presented in table no.5.2 The value of uniaxial compression strength (σ_{ci}) evaluated from the above tests was found to be 9.62 MPa. The modulus of elasticity of intact specimen (E_{ti}) has been calculated at 50% of the σ_{ci} value to account the tangent modulus. The value of E_{ti} were found as 361.17 MPa.

Table 5.2: values of stress and strain for intact POP specimen

POP intact specimen details for UCS test

Length of specimen = 76mm

Diameter of specimen = 38mm

Cross sectional area of the specimen = 1134 Sqmm

Axial strain, ε_a(%)	Uniaxial compressive strength, σ_{ci} (MPa)
0	0
0.658	2.03
1.316	3.96
1.974	5.45
2.632	7.81
3.289	9.41
3.947	9.62
4.605	9.52

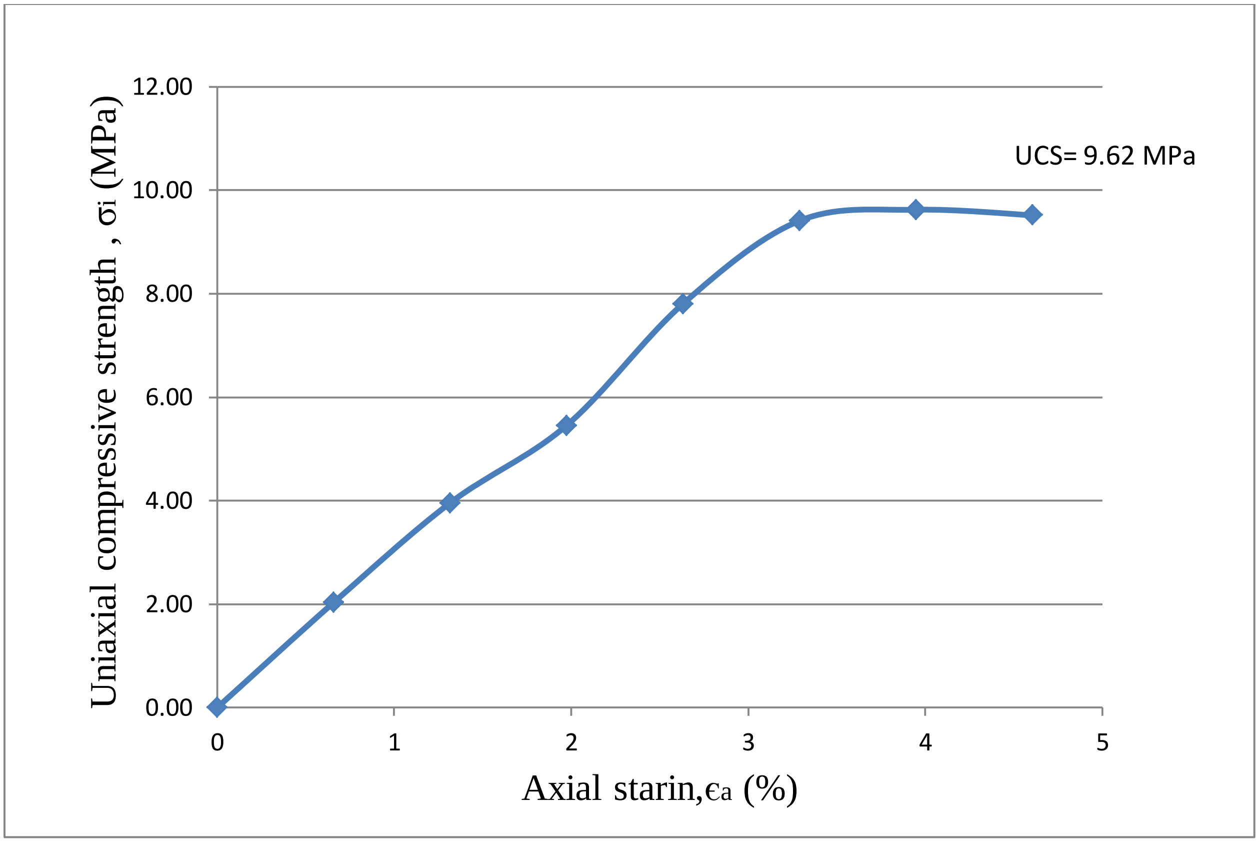


Fig. 5.6 : Axial strain vs. stress for uniaxial compressive strength of POP intact specimen

Table. 5.3: Engineering properties of plaster of Paris obtained from the UCS test and direct shear test respectively

Sl No.	Property/Parameter	Values
1	Uniaxial compressive strength, σ_{ci} (MPa)	9.62
2	Tangent modulus, (E_{ti}) (MPa)	361.17
3	Cohesion intercept, c_j (MPa)	0.178
4	Angle of friction, Φ_j (degree)	39

5.4 Experiment conducted for jointed specimen of plaster of Paris

- Strength criteria

The uniaxial compressive strength of intact specimens obtained from the test results has already been found out. In similar manner, the uniaxial compressive strength (σ_{cj}) as well as modulus of elasticity (E_{tj}) for the jointed specimens was evaluated after testing the jointed specimens. In this case, the jointed specimens are placed inside a rubber membrane before testing, to avoid slippage along the critical joints. After obtaining the values of (σ_{cj}) and E_{ti} for different orientations (β) of joints, it was observed that the jointed specimens exhibit minimum strength when the joint orientation angle was at 30º and maximum when angle was 90° The values of (σ_{cr}) for different orientation angle (β) were obtained with the help of the following relationship:

$$\sigma_{cr}=\sigma_{cj}/\sigma_{ci} \quad (5.41)$$

The values of joint factor (Jf) were evaluated by using the relationship:

$$Jf = Jn / (n*r) \quad (5.42)$$

Arora (1987) has suggested the following relationship between Jf and σcr as,

$$\sigma cr = e^{-0.008*Jf} \quad (5.43)$$

Arora (1987) has suggested the following relationship between Jf and Er as,

$$\mathrm{Er} = \mathrm{e}^{-1.15*10^{-2}\ *\mathrm{Jf}} \quad (5.44)$$

Padhy (2005) has suggested the following relationship between Jf and σcr as,

$$\sigma cr = e^{-0.09*Jf} \quad (5.45)$$

Padhy (2005)has suggested the following relationship between Jf and Er as,

$$\mathrm{Er} = \mathrm{e}^{-1.25*10^{-2}\ *\mathrm{Jf}} \quad (5.46)$$

Table. 5.4: values of J_n, J_f, σ_{cj}, σ_{cr} for POP jointed specimens (single joint)

Joint type in degrees	J_n	n	r= $\tan\Phi_j$	J_f $=J_n/(n*r)$	σ_{cj} (MPa)	$\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987) $\sigma_{cr}=e^{-0.008*Jf}$	Predicted Padhy(2005) $\sigma_{cr}=e^{-0.09*Jf}$
0	13	0.810	0.809	19.839	7.230	0.7516	0.85325	0.16772
10	13	0.460	0.809	34.933	6.910	0.7173	0.75619	0.04311
20	13	0.105	0.809	153.040	4.170	0.4335	0.29396	0.00000
30	13	0.046	0.809	349.331	1.920	0.1996	0.06114	0.00000
40	13	0.071	0.809	226.327	3.310	0.3441	0.16355	0.00000
50	13	0.306	0.809	52.514	6.420	0.6674	0.65697	0.00886
60	13	0.465	0.809	34.557	7.060	0.7339	0.75846	0.04459
70	13	0.634	0.809	25.346	7.380	0.7672	0.81647	0.10217
80	13	0.814	0.809	19.741	7.590	0.7890	0.85391	0.16920
90	13	1.000	0.809	16.069	8.660	0.9116	0.87937	0.23546

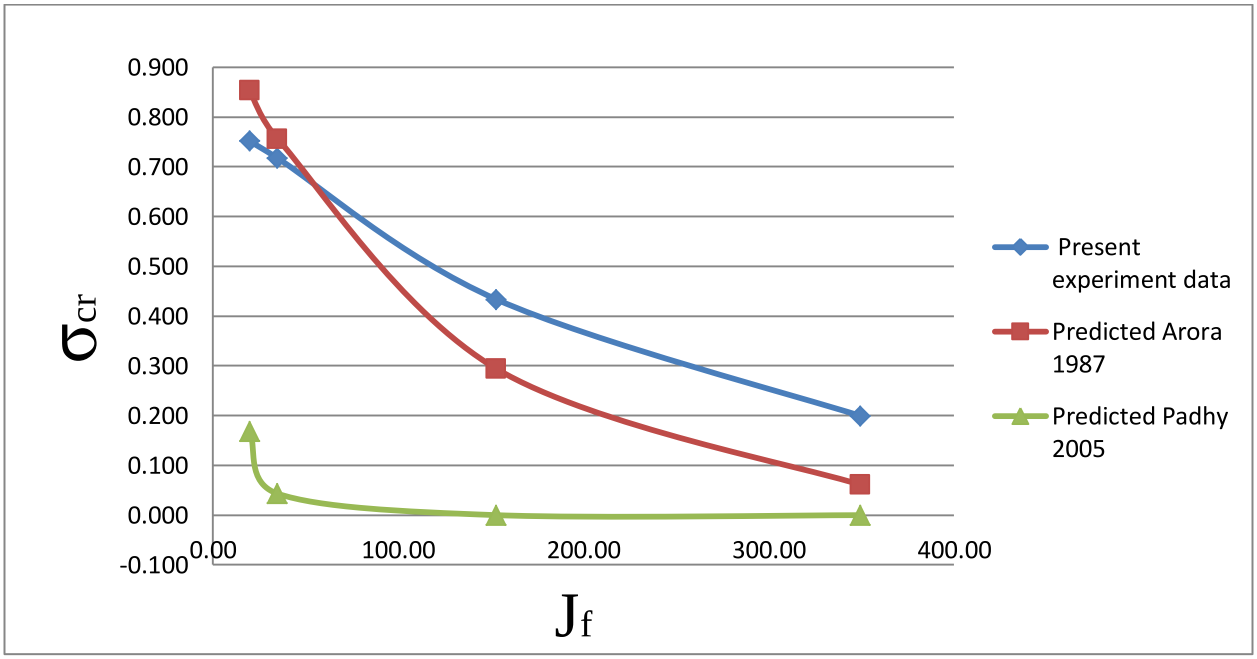


Fig .5.7: Joint factor vs. compressive strength ratio (POP Single joint specimen)

Table.5.5 : values of J_n, J_f, σ_{cj}, σ_{cr} for POP jointed specimens (double joint)

Joint type in degrees	J_n	n	r= $\tan\Phi_j$	J_f $=J_n/(n*r)$	σ_{cj} (MPa)	$\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987) $\sigma_{cr}=e^{-0.008*Jf}$	Predicted Padhy(2005) $\sigma_{cr}=e^{-0.09*Jf}$
10	26	0.460	0.809	69.866	5.630	0.5852	0.57182	0.00186
20	26	0.105	0.809	306.080	2.250	0.2339	0.08641	0.00000
30	26	0.046	0.809	698.662	0.640	0.0665	0.00374	0.00000
40	26	0.071	0.809	452.654	1.710	0.1778	0.02675	0.00000
50	26	0.306	0.809	105.028	4.810	0.5000	0.43162	0.00008
60	26	0.465	0.809	69.115	5.670	0.5894	0.57527	0.00199
70	26	0.634	0.809	50.692	6.840	0.7110	0.66662	0.01044
80	26	0.814	0.809	39.482	7.060	0.7339	0.72916	0.02863
90	26	1.000	0.809	32.138	7.810	0.8119	0.77329	0.05544

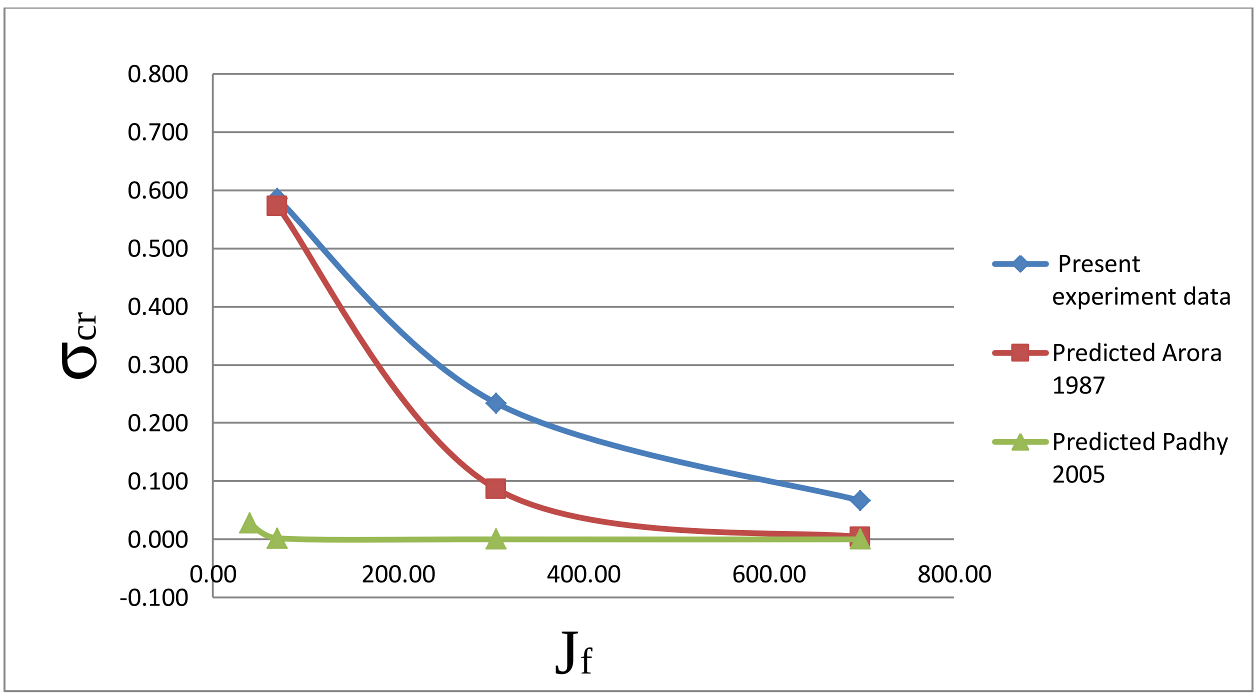


Fig. 5.8: Joint factor vs. compressive strength ratio (POP double joint specimen)

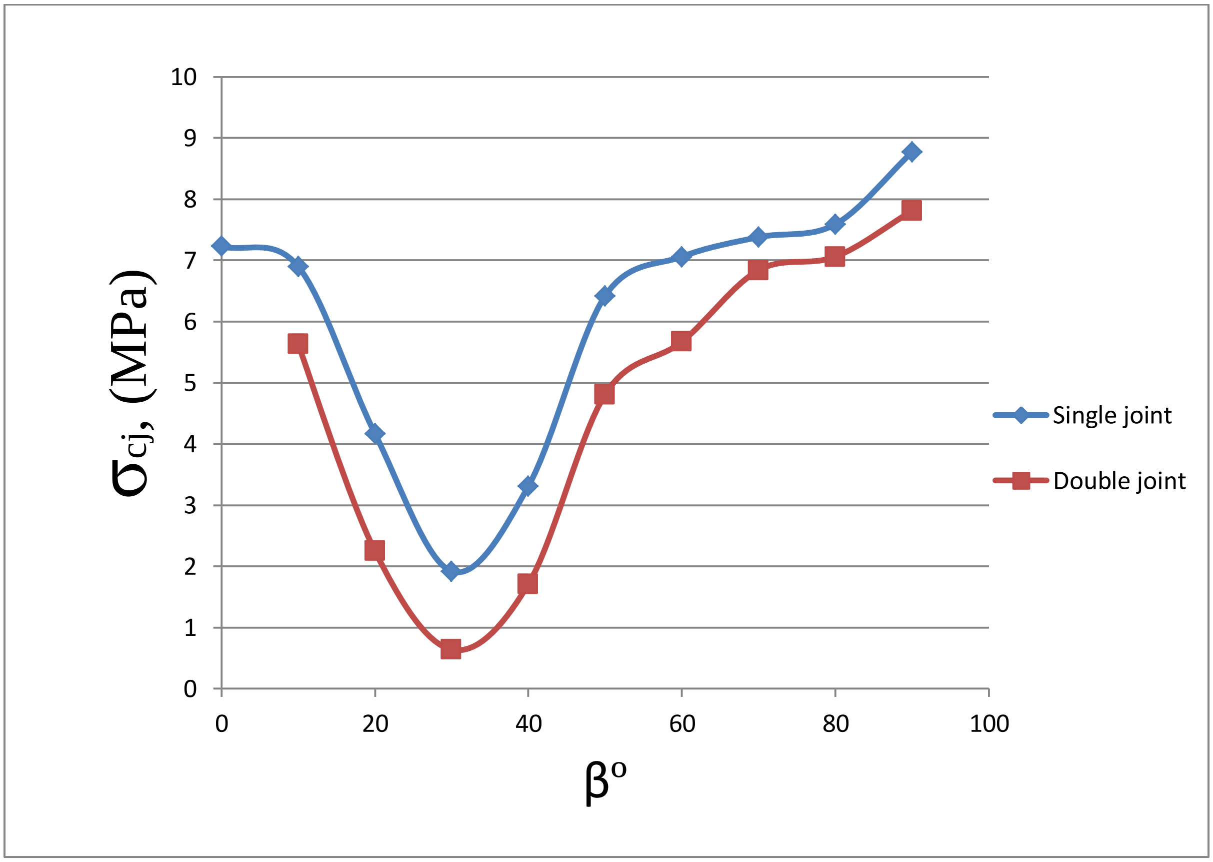


Fig.5.9: Orientation angle (β˚) vs. Uniaxial compressive strength, σ_{cj}(MPa) of POP jointed specimen represents the nature of compressive strength anisotropy.

Table.5.6: values of J_f , E_{tj}, E_r for POP jointed specimens (single joint)

Joint type in degrees	J_n	n	r = $\tan\Phi_j$	J_f $=J_n/(n*r)$	E_{tj} (MPa)	E_r $=E_{tj}/E_{ti}$	Predicted Arora(1987) Er= $e^{-1.15*10^{-2}\ Jf}$	Predicted Padhy(2005) Er= $e^{-1.25*10^{-2}\ Jf}$
0	13	0.810	0.809	19.839	312.90	0.866	0.7960	0.7804
10	13	0.460	0.809	34.933	308.30	0.854	0.6692	0.6462
20	13	0.105	0.809	153.040	104.50	0.289	0.1721	0.1476
30	13	0.046	0.809	349.331	29.94	0.083	0.0180	0.0127
40	13	0.071	0.809	226.327	73.82	0.204	0.0741	0.0591
50	13	0.306	0.809	52.514	203.68	0.564	0.5467	0.5187
60	13	0.465	0.809	34.557	239.65	0.664	0.6721	0.6492
70	13	0.634	0.809	25.346	234.67	0.650	0.7472	0.7285
80	13	0.814	0.809	19.741	263.86	0.731	0.7969	0.7813
90	13	1.000	0.809	16.069	312.96	0.867	0.8313	0.8180

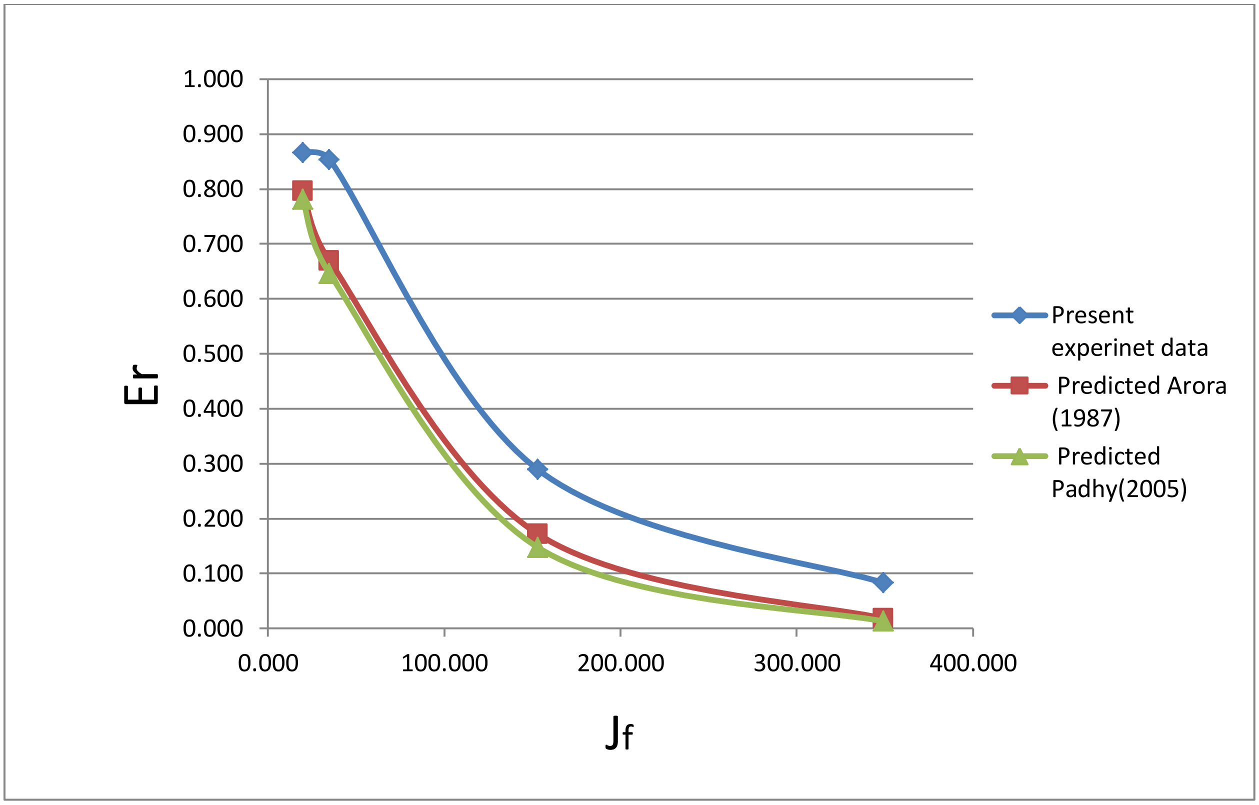


Fig. 5.10: Joint factor vs. modular ratio (POP single joint specimen)

Table.5.7: values of J_f ,E_{tj}, E_r for POP jointed specimens (double joint)

Joint type in degrees	J_n	n	r = $\tan\Phi_j$	$J_f = J_n/(n*r)$	E_{tj} (MPa)	$E_r = E_{tj}/E_{ti}$	Predicted Arora(1987) Er= $e^{-1.15*10^{-2}\ Jf}$	Predicted Padhy(2005) Er= $e^{-1.25*10^{-2}\ Jf}$
10	26	0.460	0.809	69.866	225.00	0.623	0.4478	0.4176
20	26	0.105	0.809	306.080	63.00	0.174	0.0296	0.0218
30	26	0.046	0.809	698.662	18.46	0.051	0.0003	0.0002
40	26	0.071	0.809	452.654	27.44	0.076	0.0055	0.0035
50	26	0.306	0.809	105.028	96.73	0.268	0.2988	0.2691
60	26	0.465	0.809	69.115	171.13	0.474	0.4517	0.4215
70	26	0.634	0.809	50.692	196.00	0.543	0.5582	0.5307
80	26	0.814	0.809	39.482	199.00	0.551	0.6351	0.6105
90	26	1.000	0.809	32.138	269.00	0.745	0.6910	0.6692

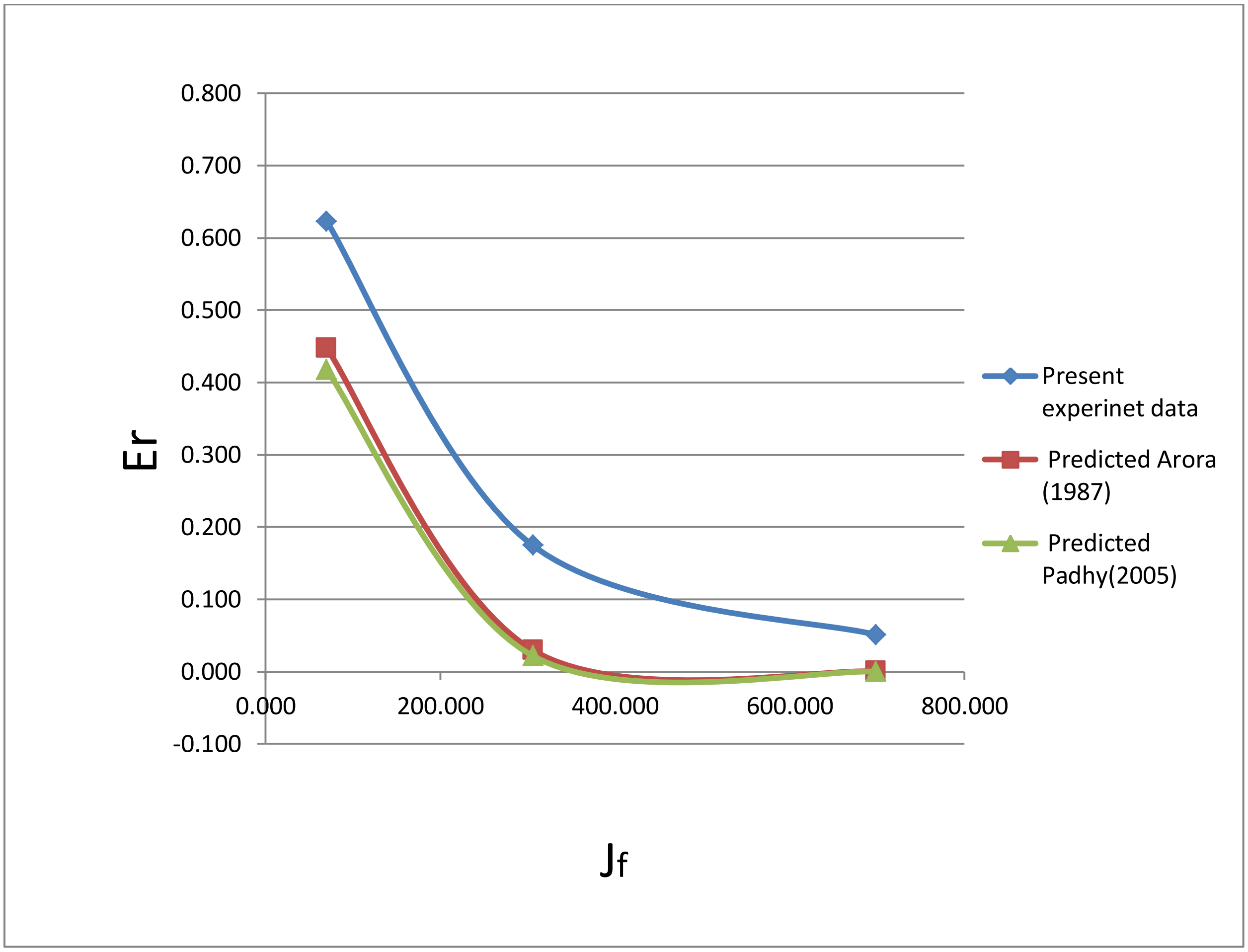


Fig.5.11 : Joint factor vs. modular ratio(POP double joint specimen)

5.5 Direct shear test results of POP-sand mix test specimen

The roughness parameter (r) which is the tangent value of the friction angle (Φj) was obtained from the direct shear test conducted at different normal stresses. The value of cohesion (c_j) for jointed specimens of plaster of Paris , sand mix specimen has been found as 0.182 MPa and value of friction angle (Φ_j) found as 41°. Hence the roughness parameter ($r = \tan\Phi_j$) comes to be 0.839 for the specimens of plaster of Paris tested.

Table. 5.8: Values of shear stress for different values of normal stress for POP-sand jointed specimen in direct shear stress test.

Normal stress ,σ_n (MPa)	Shear stress, τ (MPa)
0.049	0.328
0.098	0.462
0.147	0.581

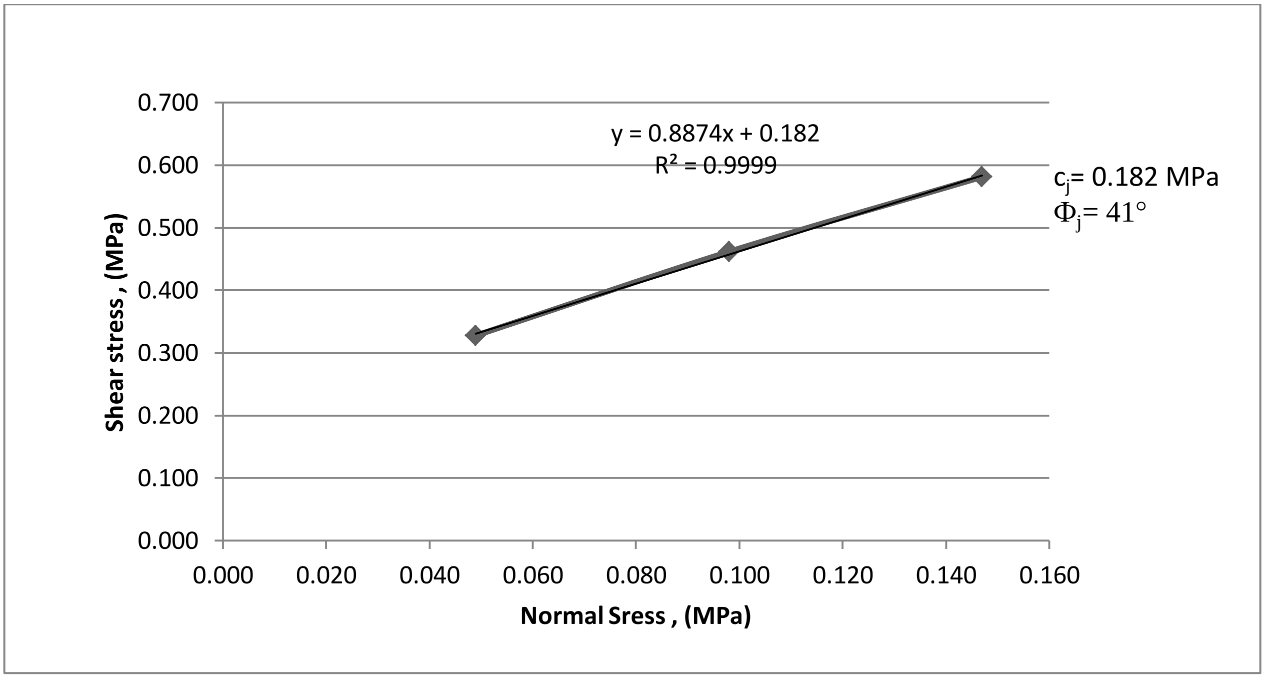


Fig. 5.12: Normal stress vs.. shear stress of POP-sand mix specimen

5.6 Uniaxial compression test results of POP- sand mix intact specimen

The variations of the stress with strain as obtained by uniaxial compression strength test for the intact specimen of plaster of Paris and sand mix and its corresponding stress Vs.. strain values are presented in table no.. The value of uniaxial compression strength (σ_{ci}) evaluated from the above tests was found to be 10.37 MPa. The modulus of elasticity of intact specimen (E_{ti}) has been calculated at 50% of the σ_{ci} value to account the tangent modulus. The value of E_{ti} were found as 402.84 MPa.

Table.5.9: values of stress and strain of POP-sand mix intact specimen

POP intact specimen details for UCS test

Length of specimen = 76mm

Diameter of specimen = 38mm

Cross sectional area of the specimen = 1134 Sqmm

Axial strain, ε_a(%)	Uniaxial compressive strength, σ_{ci} (MPa)
0	0.00
0.658	2.25
1.316	3.74
1.974	5.67
2.632	7.81
3.289	9.41
3.947	10.37
4.605	10.27
5.263	10.16

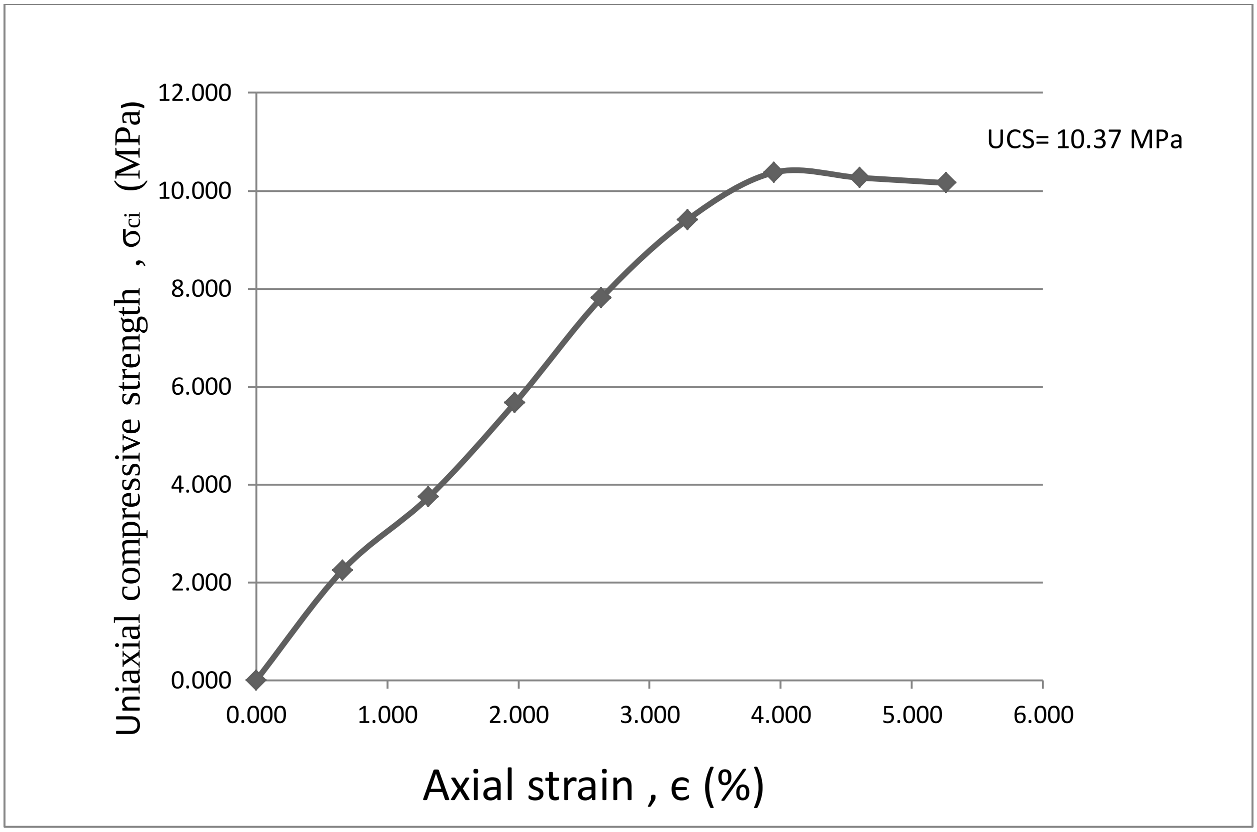


Fig.5.13: Axial strain vs. stress for uniaxial compressive strength of POP-sand mix intact specimen .

Table. 5.10: Engineering properties of POP-sand mix specimen obtained from the UCS test and direct shear test respectively.

Sl No.	Property/Parameter	Values
1	Uniaxial compressive strength, σ_{ci} (MPa)	10.37
2	Tangent modulus, (E_{ti}) (MPa)	402.84
3	Cohesion intercept, c_j (MPa)	0.182
4	Angle of friction, Φ_j (degree)	41

5.7 Experiment conducted for jointed specimen of POP-sand mix specimen

- Strength criteria

The uniaxial compressive strength of intact specimens obtained from the test results has already been found out. In similar manner, the uniaxial compressive strength (σ_{cj}) as well as modulus of elasticity (E_{tj}) for the jointed specimens was evaluated after testing the jointed specimens. In this case, the jointed specimens are placed inside a rubber membrane before testing, to avoid slippage along the critical joints. After obtaining the values of (σ_{cj}) and E_{ti} for different orientations (β) of joints, it was observed that the jointed specimens exhibit minimum strength when the joint orientation angle was at 30º and maximum when angle was 90° The values of (σ_{cr}) for different orientation angle (β) were obtained with the help of the following relationship:

$$\sigma_{cr}=\sigma_{cj}/\sigma_{ci} \quad (5.71)$$

The values of joint factor (Jf) were evaluated by using the relationship:

$$Jf = Jn / (n*r) \quad (5.72)$$

Arora (1987) has suggested the following relationship between Jf and σcr as,

$$\sigma cr = e^{-0.008*Jf} \quad (5.73)$$

Arora (1987) has suggested the following relationship between Jf and Er as,

$$\mathrm{Er} = \mathrm{e}^{-1.15*10^{-2}*\mathrm{Jf}} \quad (5.74)$$

Padhy (2005) has suggested the following relationship between Jf and σcr as,

$$\sigma cr = e^{-0.09*Jf} \quad (5.75)$$

Padhy (2005)has suggested the following relationship between Jf and Er as,

$$\mathrm{Er} = \mathrm{e}^{-1.25*10^{-2}*\mathrm{Jf}} \quad (5.76)$$

Table. 5.11: values of J_n, J_f, σ_{cj},,σ_{cr} for POP-sand mix single joint specimens

Joint type in degrees	J_n	n	r= $\tan\Phi_j$	J_f $=J_n/(n*r)$	σ_{cj} (MPa)	σ_{cr}= σ_{cj}/σ_{ci}	Predicted Arora(1987) $\sigma_{cr}=e^{-0.008*Jf}$	Predicted Padhy(2005) $\sigma_{cr}=e^{-0.09*Jf}$
0	13	0.810	0.869	18.469	8.03	0.877	0.86265	0.18972
10	13	0.460	0.869	32.521	7.23	0.794	0.77092	0.05356
20	13	0.105	0.869	142.474	3.96	0.382	0.31989	0.00000
30	13	0.046	0.869	325.211	2.14	0.206	0.07415	0.00000
40	13	0.071	0.869	210.700	3.74	0.361	0.18533	0.00000
50	13	0.306	0.869	48.888	6.1	0.588	0.67631	0.01228
60	13	0.465	0.869	32.171	7.06	0.681	0.77308	0.05528
70	13	0.634	0.869	23.596	8.45	0.815	0.82798	0.11960
80	13	0.814	0.869	18.378	9.09	0.877	0.86327	0.19128
90	13	1.000	0.869	14.960	9.52	0.918	0.88721	0.26018

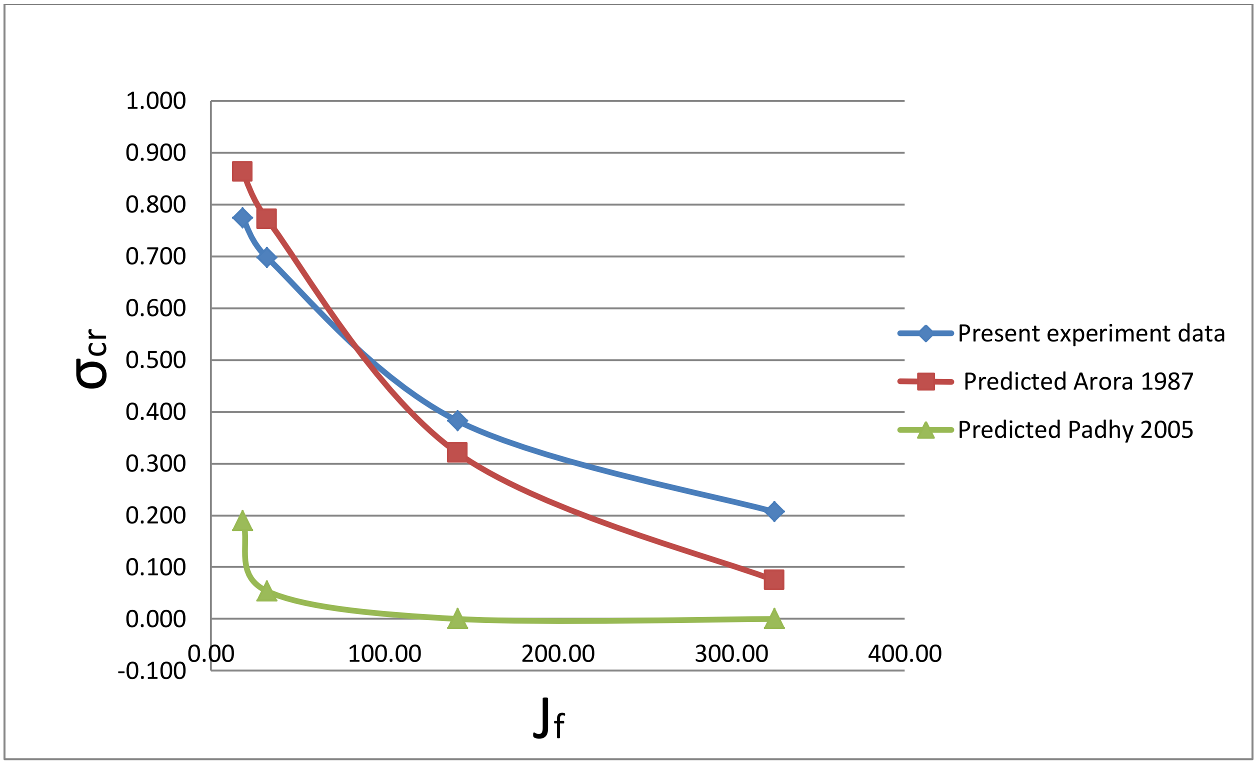


Fig .5.14 : Joint factor vs.. compressive strength ratio for POP-sand mix single joint specimen

Table.5.12: values of J_n, J_f, σ_{cj}, σ_{cr} for POP-sand mix double joint specimens

Joint type in degrees	J_n	n	r= $\tan\Phi_j$	J_f $=J_n/(n*r)$	σ_{cj} (MPa)	$\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987) $\sigma_{cr}=e^{-0.008*Jf}$	Predicted Padhy(2005) $\sigma_{cr}=e^{-0.09*Jf}$
10	26	0.460	0.869	65.042	6.12	0.681	0.59432	0.00287
20	26	0.105	0.869	284.947	2.14	0.206	0.10233	0.00000
30	26	0.046	0.869	650.423	1.6	0.154	0.00550	0.00000
40	26	0.071	0.869	421.401	2.25	0.217	0.03435	0.00000
50	26	0.306	0.869	97.776	4.91	0.473	0.45740	0.00015
60	26	0.465	0.869	64.343	5.67	0.547	0.59765	0.00306
70	26	0.634	0.869	47.192	7.06	0.681	0.68555	0.01430
80	26	0.814	0.869	36.756	8.02	0.773	0.74524	0.03659
90	26	1.000	0.869	29.919	8.34	0.804	0.78713	0.06769

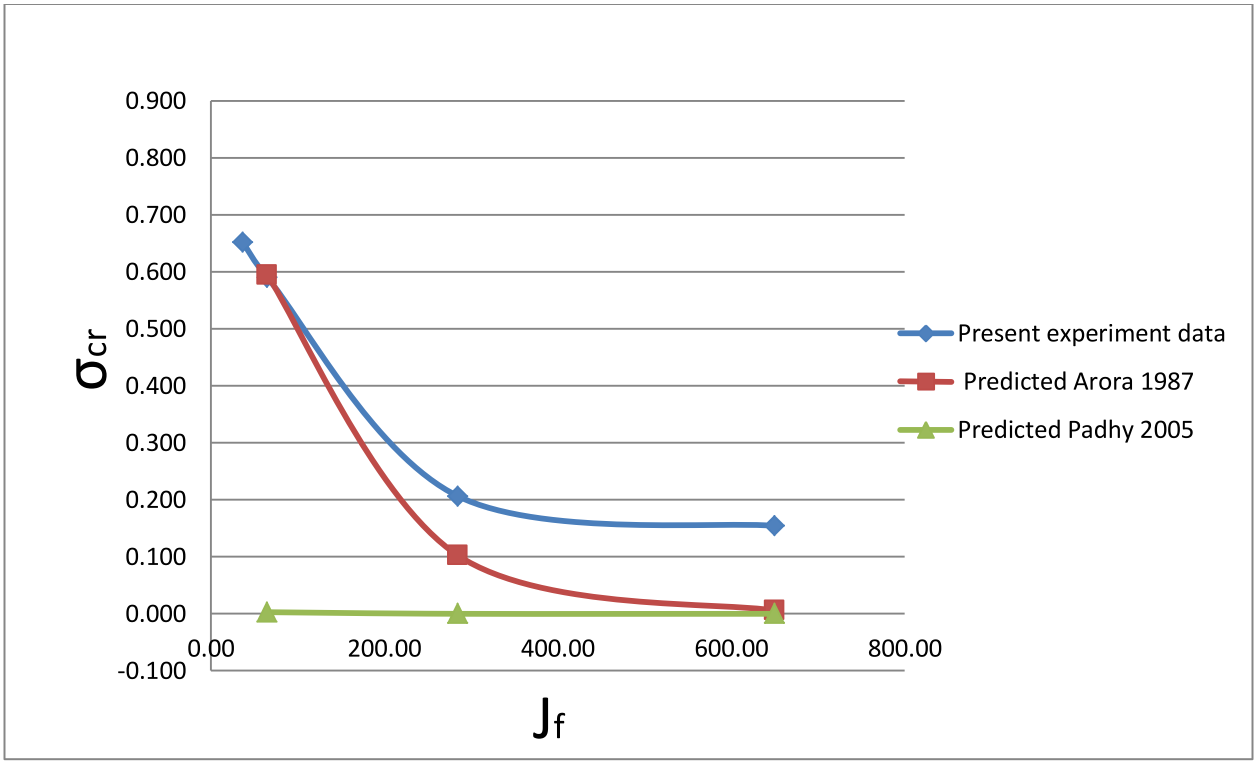


Fig .5.15 : Joint factor vs.. compressive strength ratio for POP-sand mix double joint specimen

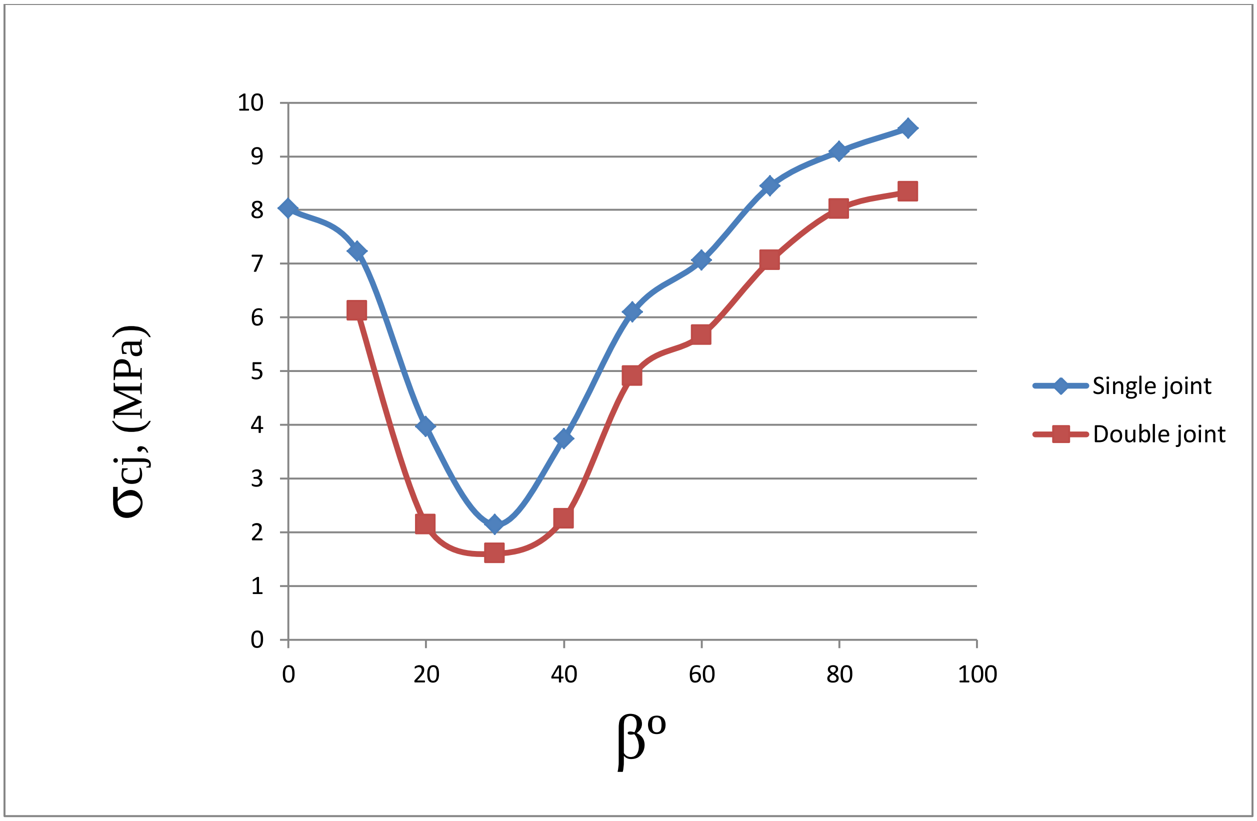


Fig.5.16: Orientation angle (β˚) vs.. Uniaxial compressive strength, σ_{cj}(MPa) of POP-sand mix jointed specimen represents the nature of compressive strength anisotropy.

Table. 5.13: Values of J_f ,E_{tj}, E_r for POP-sand mix single joint specimens

Joint type in degrees	J_n	n	r = $\tan\Phi_j$	J_f $=J_n/(n*r)$	E_{tj} (MPa)	E_r $=E_{tj}/E_{ti}$	Predicted Arora(1987) Er= $e^{-1.15*10^{-2}\ Jf}$	Predicted Padhy(2005) Er= $e^{-1.25*10^{-2}\ Jf}$
0	13	0.810	0.869	18.469	323.38	0.803	0.809	0.794
10	13	0.460	0.869	32.521	292.78	0.886	0.688	0.666
20	13	0.105	0.869	142.474	98.76	0.245	0.194	0.168
30	13	0.046	0.869	325.211	32.32	0.080	0.024	0.017
40	13	0.071	0.869	210.700	82.48	0.205	0.089	0.072
50	13	0.306	0.869	48.888	193.52	0.480	0.570	0.543
60	13	0.465	0.869	32.171	238.99	0.593	0.691	0.669
70	13	0.634	0.869	23.596	267.92	0.665	0.762	0.745
80	13	0.814	0.869	18.378	314.97	0.782	0.809	0.795
90	13	1.000	0.869	14.960	339.72	0.843	0.842	0.829

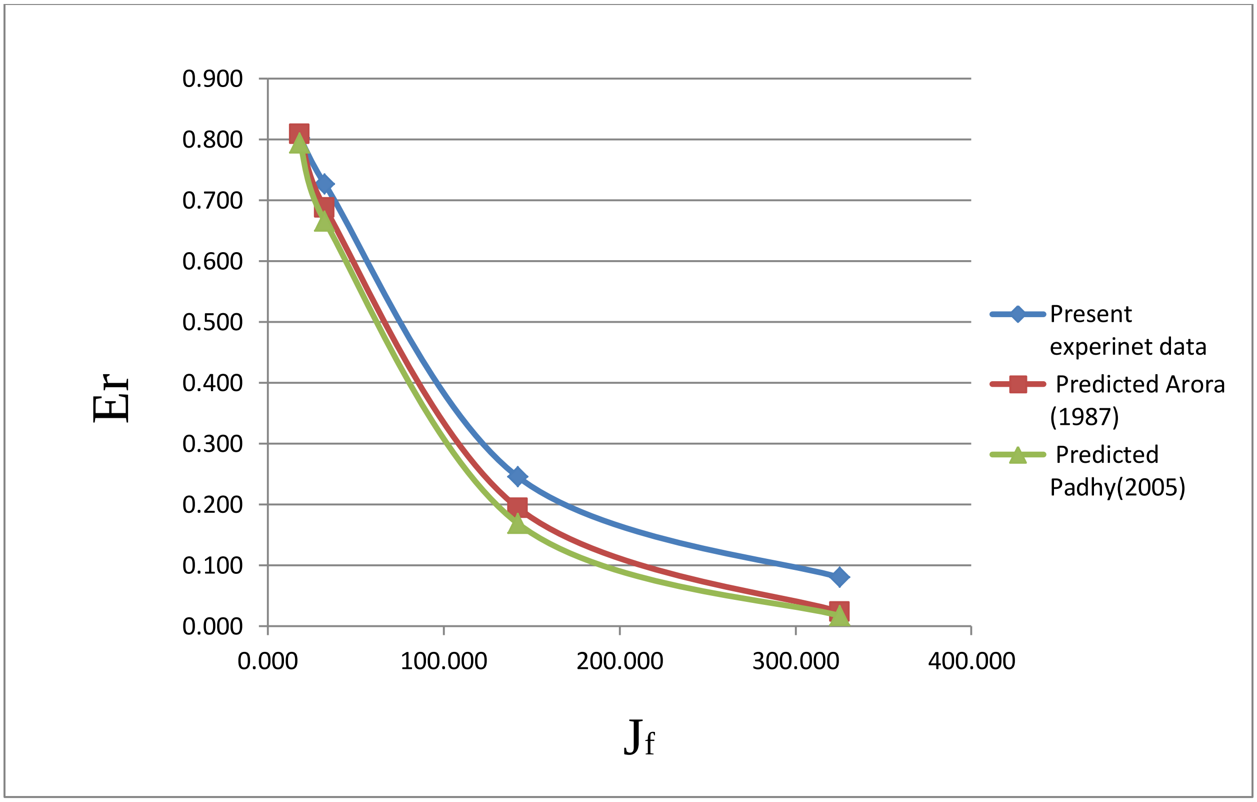


Fig. 5.17: Joint factor vs.. modular ratio for POP –sand mix single joint specimen

Table. 5.14: Values of J_f, E_{tj}, E_r for POP-sand mix double joint specimens

Joint type in degrees	J_n	n	r = $\tan\Phi_j$	$J_f = J_n/(n*r)$	E_{tj} (MPa)	$E_r = E_{tj}/E_{ti}$	Predicted Arora(1987) Er= $e^{-1.15*10^{-2}\ Jf}$	Predicted Padhy(2005) Er= $e^{-1.25*10^{-2}\ Jf}$
10	26	0.460	0.869	65.042	250.86	0.623	0.473	0.444
20	26	0.105	0.869	284.947	53.37	0.132	0.038	0.028
30	26	0.046	0.869	650.423	24.16	0.060	0.001	0.000
40	26	0.071	0.869	421.401	49.62	0.123	0.008	0.005
50	26	0.306	0.869	97.776	155.76	0.387	0.325	0.295
60	26	0.465	0.869	64.343	191.93	0.476	0.477	0.447
70	26	0.634	0.869	47.192	223.07	0.554	0.581	0.554
80	26	0.814	0.869	36.756	277.89	0.690	0.655	0.632
90	26	1.000	0.869	29.919	296.98	0.737	0.709	0.688

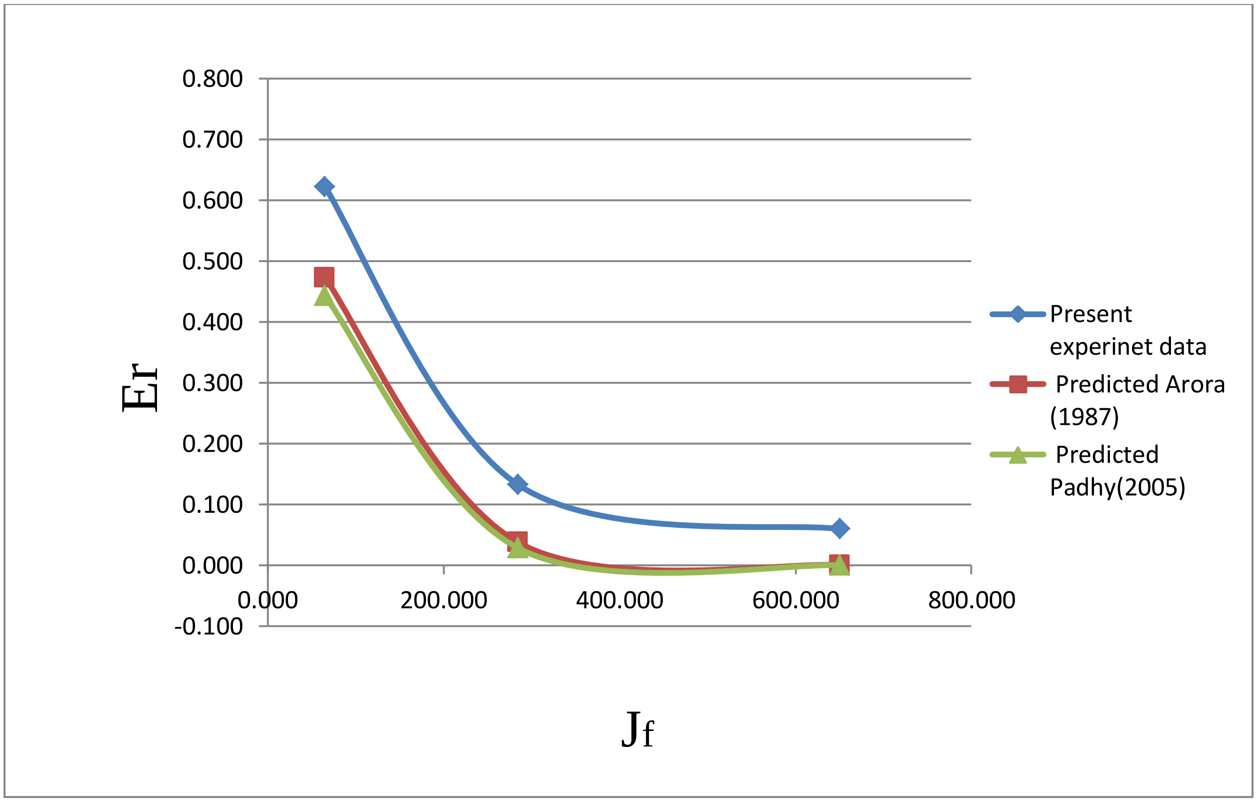


Fig. 5.18: Joint factor vs. modular ratio for POP-sand mix double joint specimen

Table.5.15: Summary of strength classification for POP intact specimen

Strength classification proposed by previous researchers				Strength classification based on present experimental study	
Reference details	Description	class	UCS range (MPa)	UCS (MPa)	Remarks
(Stapledon and ISRM 1971)	Weak	W	5-25	9.62	Weak rock
BIENIAWSKI, 1971”	Very weak	VW	2-25	9.62	Very weak
Deere and Miller, 1966”	Very low strength	E	<28	9.62	Very low strength
Ramamurthy and Arora 1994	Low strength	E	5-25	9.62	Low strength

Table.5.16: Summary of strength classification for POP-sand mix intact specimen

Strength classification proposed by previous researchers				Strength classification based on present experimental study	
Reference details	Description	class	UCS range (MPa)	UCS (MPa)	Remarks
(Stapledon and ISRM 1971)	Weak	W	5-25	10.37	Weak rock
BIENIAWSKI, 1971”	Weak	W	10-25	10.37	Weak
Deere and Miller, 1966”	Very low strength	E	<28	10.37	Very low strength
Ramamurthy and Arora 1994	Low strength	E	5-25	10.37	Low strength

Table. 5.17: Values of uniaxial compressive strength (σ_{cj}) of jointed specimen with different joint orientation angle (β°) for POP single and double joint specimen

β°	UCS single joint σ_{cj} (MPa)	UCS double joint σ_{cj} (MPa)	Decrease value (MPa)	Remarks
10	7.81	6.63	1.18	While no of joint per meter increases strength decreases (no of joint per meter for single joint is 13 and for double joint is 26)
20	4.17	2.25	1.92	
30	1.92	0.64	1.28	
40	3.31	1.71	1.6	
50	6.42	4.81	1.61	
60	7.06	5.67	1.39	
70	7.38	6.84	0.54	
80	7.59	7.06	0.53	
90	8.66	7.81	0.85	

Table. 5.18 Values of uniaxial compressive strength (σ_{cj}) of jointed specimen with different joint orientation angle (β°) for POP-sand mix specimen single and double joint.

β^{o}	UCS for single joint ,σ_{cj} (MPa)	UCS for double joint ,σ_{cj} (MPa)	Decrease in value (MPa)	Remarks
10	8.23	7.06	1.17	While no of joint per meter increases strength decreases.
20	4.49	2.14	2.35	
30	2.14	1.6	0.54	
40	3.74	2.25	1.49	
50	6.63	4.91	1.72	
60	7.27	5.67	1.6	
70	8.45	7.06	1.39	
80	9.09	8.02	1.07	
90	9.52	8.34	1.18	

Table.5.19 Values of uniaxial compressive strength ratio (σ_{cr}) of jointed specimen with different joint orientation angle (β°) for single and double joint POP specimen .

β^{o}	Single joint $\sigma_{cr} = (\sigma_{cj} / \sigma_{ci,})$,(MPa)	Double joint $\sigma_{cr} = (\sigma_{cj} / \sigma_{ci})$, (MPa)	Decrease in σ_{cr} (MPa)	Remarks
10	0.7173	0.5852	0.1321	while no of joint per meter increases strength ratio decreases.
20	0.4335	0.2339	0.1996	
30	0.1996	0.0665	0.1331	
40	0.3441	0.1778	0.1663	
50	0.6674	0.5000	0.1674	
60	0.7339	0.5894	0.1445	
70	0.7672	0.7110	0.0562	
80	0.7890	0.7339	0.0551	
90	0.9116	0.8119	0.0997	

Table. 5.20: Values of uniaxial compressive strength ratio (σ_{cr}) of jointed specimen with different joint orientation angle (β˚) for single and double joint POP-sand mix specimen.

β^o	Single joint $\sigma_{cr} = (\sigma_{cj}/\sigma_{ci,})$ (MPa)	Double joint $\sigma_{cr} = (\sigma_{cj}/\sigma_{ci})$, (MPa)	Decrease in σ_{cr} (MPa)	Remarks
10	0.794	0.681	0.113	Strength ratio decreases while no of joint per meter increases
20	0.382	0.206	0.176	
30	0.206	0.154	0.052	
40	0.361	0.217	0.144	
50	0.588	0.473	0.115	
60	0.681	0.547	0.134	
70	0.815	0.681	0.134	
80	0.877	0.773	0.104	
90	0.918	0.804	0.114	

Table . 5.21: Values of Elastic modulus ratio (E_r) of jointed specimen with different joint orientation angle (β˚) for single and double joint POP specimen .

β°	Single joint E_r (MPa)	Double joint E_r , (MPa)	Reduction in E_r , (MPa)	Remarks
10	0.854	0.623	0.231	Value of modulus ratio decreases while no of joint per meter increases
20	0.289	0.174	0.115	
30	0.083	0.051	0.032	
40	0.204	0.076	0.128	
50	0.564	0.268	0.296	
60	0.664	0.474	0.19	
70	0.65	0.543	0.107	
80	0.731	0.551	0.18	
90	0.867	0.745	0.122	

Table 5.22: Values of Elastic modulus ratio (E_r) of jointed specimen with different joint orientation angle (β˚) for single and double joint POP-sand mix specimen .

βº	Single joint E_r , MPa)	Double joint E_r ,(MPa)	Reduction in E_r , (MPa)	Remarks
10	0.886	0.623	0.263	Value of modulus ratio decreases while no of joint per meter increases
20	0.245	0.132	0.113	
30	0.08	0.06	0.02	
40	0.205	0.123	0.082	
50	0.48	0.387	0.093	
60	0.593	0.476	0.117	
70	0.665	0.554	0.111	
80	0.782	0.69	0.092	
90	0.843	0.737	0.106	

CHAPTER -6

Conclusions

1. The cohesion (cj) and friction angle (Φj) for plaster of Paris specimen was found to be 0.178 MPa and 39^0, whereas for POP-sand mix specimen was found to be 0.182 MPa and 41° respectively.
2. The Uniaxial compressive strength of POP and POP-sand mix intact specimens were found to be 9.62MPa and 10.37MPa respectively.
3. Specimens tested , which has fall under the low strength rock category (Ref. Table no- 5.15 , 5.16)
4. The strength of jointed specimen depends on the joint orientation β with respect to the direction of major principal stress, and uniaxial compressive strength has found maximum at 0° & 90° and minimum at 30° (Ref.Table no- 5.4, 5.5, 5.11, 5.12).
5. Empirical relationship of Arora (1987) and Padhy (2005) have been used to predicting the result of strength ratio and modulus ratio of jointed rock mass and it seems that which is almost closer with the present experimental results of strength ratio and modulus ratio.

[Empirical relationship of Arora (1987), $\sigma cr = e^{-0.008*Jf}$ and $Er = e^{-1.15*10^{-2}*Jf}$]

[Empirical relationship of Padhy (2005), $\sigma cr = e^{-0.09*Jf}$ and $Er = e^{-1.25*10^{-2}*Jf}$]

6. The values of modulus ratio ($E_r=E_{tj}/E_{ti}$) also depends on the joint orientation β. The modulus ratio is least at 30° and maximum at 0°& 90°.(Ref. Table no-5.6, 5.7, 5.13, 5.14)

7. The values of compressive strength ratio ($\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$) also depends on the joint orientation β. This ratio is least at 30° and maximum at 0° & 90° (Ref. Table no-5.4 , 5.5 , 5.11, 5.12).

8. Strength , elastic modulus increases while POP mixes with sand

9. Strength decreases with the increases of no of joints.

10. One may predict the strength of jointed rock mass by knowing the uniaxial compressive strength and joint factor using the relationship between σ_{cr} and J_f as suggested by Arora (1987), and Padhy (2005)

CHAPTER-7

Scope of further study

1. Strength and deformation behavior of jointed specimen can be studied by introducing multiple joints in varying orientation angle.

2. Strength and deformation behavior of jointed specimens can be studied with the joint by gouge filled materials.

3. Studies for variation of uniaxial compressive strength on L/D ratio.

4. Different Software can be used to analyze the Experimental result

CHAPTER-8

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